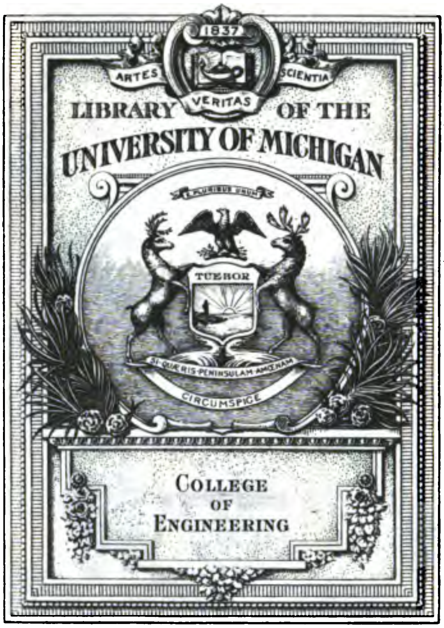

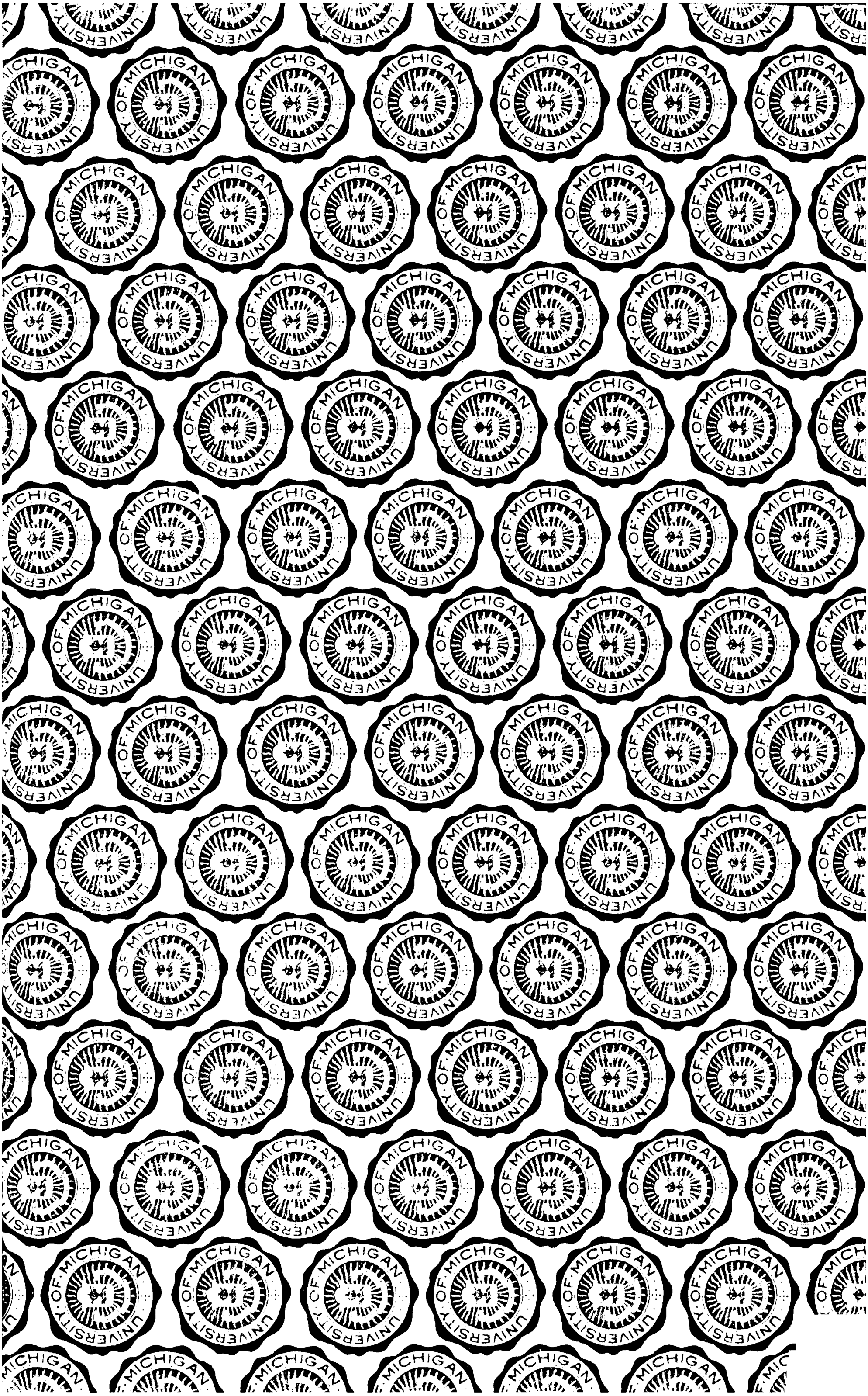
UNIVERSITY OF MICHIGAN

ALLOWABLE PRESSURES
ON
DEEP FOUNDATIONS

BY

ELMER LAWRENCE CORTHELL
D.Sc., M.Inst.C.E.

NEW YORK
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43 and 45 EAST NINETEENTH STREET
1907

ERRATA

In Plate No. **2**, Appendix A, colûmn 15, "Maximum pressure in same terms," Case No. 11, Chimney at Lawrence, Massachusetts, change 24 tons to 2.4 tons.

River at depths ranging from 15 metres (49·2 feet), the minimum, to 23·5 metres (71·1 feet) below low water, the bed of the river being 6·50 metres (21·3 feet) below low water. The piers were to be spaced 16 metres (52·5 feet), centre to centre, the intermediate construction being horizontal metallic lintels (*linteaux*) with arches in steel concrete abutting against the piers; the lintels were to be placed 1·5 metre (4·9 feet) above low water—the zero.

The original plans presented contemplated piers 20 metres (65·6 feet) centre to centre, with a maximum pressure of about 8 kilograms per square centimetre (7·3 tons of 2,240 lbs. per square foot).

The individual views of the members of the Board ranged from

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[1] An Abstract of this Paper was published in the Proceedings of the Inst. C.E., vol. clxv. Session 1905-1906. Part III.

an allowable pressure of 3 kilograms per square centimetre (2·7 tons per square foot) to 5·5 kilograms per square centimetre (5 tons per square foot).

The contractor, Mr. Hersent, who was called in to give his reasons for the large pressure proposed, stated that on other important works, notably at Bordeaux, he had placed weights equal to those proposed by him on material with no greater resisting power than that to be met with at Rozario.

However, the Board finally decided upon 3½ kilograms per square centimetre (3·2 tons per square foot), which decision required the placing of the piers 16 metres (52·5 feet) apart and to enlarge their bases, considerably increasing the cost of the work.

The writer was not satisfied with the pressure to be allowed. He soon afterwards sent a cablegram to New York to learn the opinion of Mr. G. S. Morison and Mr. Alfred Noble, with both of whom he had been associated in deep-foundation work on the Mississippi, Missouri, Ohio and other rivers. Mr. Morison was absent in Mexico and not accessible, but Mr. Noble's reply was to the effect that 5 tons per square foot (5·5 kilograms per square centimetre) was an allowable pressure in deep foundations on the rivers before mentioned.

The writer referred the matter again to the members of the Board individually—it having adjourned *sine die*, and a majority gave their opinion in favour of 5 tons (5·5 kilograms), and in the contract made later by the Minister of Public Work this pressure was fixed upon as a maximum with 3·34 tons (3·65 kilograms) as a mean.

The wide difference of opinion among experts disclosed by this discussion determined the writer to make an exhaustive investigation for the benefit of the profession—as soon as he could find the time to do so.

On his return to the United States in the fall of 1902, he endeavoured to find some young engineer who would, out of love to his profession, assist him in this work, but he was unable to make such an arrangement.

He then decided to employ an expert at his own expense. Mr. Charles R. Wychoff, Jr., at that time assistant to Professor Burr of Columbia University, was selected, and the plan of operation was discussed with him and decided upon.

A circular-letter was drawn up, printed and sent to about 300 engineers in various countries. In order that the scope of the investigation may be seen, a copy of this circular-letter is given following which fully explains itself.

COPY OF CIRCULAR-LETTER, DATED NEW YORK, DECEMBER 5TH, 1902.

DEAR SIR,

Recently the writer, in the course of his professional experience, was obliged to give his opinion upon the allowable pressure upon the foundation material of an important structure.

His own experience, and that of the others, obtainable at the time, either in books or reports, was not sufficient to give all the information necessary on which to base an opinion. After some correspondence with other engineers, he has decided to make a thorough investigation of the subject for the general benefit of the profession, and has employed a competent engineer to compile data under the writer's general direction.

If you can aid in this investigation the writer will be glad to give you the final results, either directly or through a professional paper which he may contribute to some engineering society.

Enclosed is a blank form to be filled out with as many examples as possible of cases that have come under your own observation, or that you can give from your records of works of others. Space is provided between the subjects for the data in reference to several works. In order to avoid confusion, each work should be numbered and the numbers carried through the entire form.

As the writer is to be absent for the next five months on an extended professional tour, he requests that the data be sent to the Engineer engaged for the work, "Mr. CHARLES R. WYCHOFF, Jr., Assistant to Professor Civil Engineering, Columbia University, New York City, New York, U.S.A."

The writer is very thankful in advance for any assistance you may give him in this investigation.

I am,

Yours truly,

(Signed) E. L. CORTHELL.

PRESSURES ON DEEP FOUNDATIONS.

BRIDGES, DAMS, PORT DIKES, QUAY WALLS AND PIERS, BUILDINGS, LIGHT-HOUSES, CHIMNEYS, MONUMENTS, ETC.

No.

1.—Country.

2.—Locality.

3.—Date of construction.

4.—Name of structure.

5.—Materials for the same.

6.—Depth of the foundations below the bed of the river or harbour, or below the surface of the ground.

7.—Character of the material passed through.

8.—Method of sinking.

9.—Character of the material on which the structure rests, in as much detail as possible.

10.—Shape and area of the base.

11.—The volume of the mass below low water, in cubic yards or cubic metres.

12.—Total weight of the structure at low water, in tons of 2,000 lbs., or kilometric tons.

No.
13.—Fatigue weight upon foundations.
14.—Average pressure in tons of 2,000 lbs. per square foot, or in kilograms per square centimetre, friction neglected.
15.—Maximum pressure in same terms.
16.—Frictional resistance of the sides, expressed in pounds per square foot or kilograms per square metre.
17.—Settlement, if any, and how much, and how long continued before reaching a state of rest.
18.—If settlement caused the partial or entire destruction of the work, state how it occurred.
19.—Date when information is given.
20.—Authority.
21.—Remarks.

Mr. Wychoff was to receive and arrange the matter sent in, in tabular form, placing in twenty-one columns the information asked for in the twenty-one questions of the circular. In addition to this original data which those addressed might send in, he was to make a thorough study of all published data to be found in periodicals, and professional papers presented to engineering organizations—in fact, he was to exhaust the subject.

The writer himself turned over to Mr. Wychoff all the data on this subject which he had collected through his long experience on constructive works, and then left the work in the hands of Mr. Wychoff, he himself going on a long tour of lectures on Argentine in the United States and Mexico, being absent six months. Those lectures before universities, technical schools, engineering societies and scientific bodies, gave him an opportunity to personally explain to engineers the nature of the investigation into this subject of pressures on foundations.

In one respect the writer has been disappointed; not one-tenth of those addressed by the circular-letter responded by sending useful data—many were too busy to attend to the request, many had no data of value, etc., etc., so that only about thirty replies of any value were received.

However, the tables which accompany this Paper give data more or less complete in reference to 178 works, and it is hoped that they will be found useful in solving the problems continually arising in reference to pressure on foundations.

The deficient feature, as will be seen by an inspection of the tables, is the dearth of information in reference to pressures upon the material, which, it is hoped, engineers who had charge of the work during construction, or who have charge of the records or access to them, will supply in the discussion of this Paper, in order to finally present as large an array of facts as possible.

The tables need, no doubt, to be corrected in some particulars; also it has been difficult to ascertain in all cases whether the pressures are mean or maximum, and whether those obtained are really fatigue pressures or include buoyancy of the surrounding water and frictional resistance of the sides of the constructions.

The eight tables appear as Appendix A. Some data in regard to pressures and friction, found in an admirable report on the proposed Tampico Custom House Wharf, Mexico, by Messrs. A. J. Tullock (deceased) and Alfred Noble, are added as Appendix B.

The very full notice of the investigation by Mr. Wychoff, comprising 245 pages of manuscript, are added for further detailed information as Appendix C.

The writer will now make a brief analysis of the tables, including the data in the Tampico Report.

This analysis is based on the various classes of material so far as they could be ascertained and classified.

The Pressures of Stable Structures on Fine Sand range from 2·25 tons of 2,000 lbs. to 5·80 tons, with an average of 4·5 tons with ten examples.

On Coarse Sand and Gravel from 2·40 tons to 7·75 tons, with an average of 5·1 tons with thirty-three examples.

On Sand and Clay from 2·5 tons to 8·5 tons, with an average of 4·9 tons with ten examples.

On Alluvium and Silt from 1·5 to 6·2 tons, with an average of 2·9 tons with seven examples.

On Hard Clay from 2·0 tons to 8·0 tons, with an average of 5·08 tons with sixteen examples.

On Hard Pan from 3·0 tons to 12·0 tons, with an average of 8·7 tons with five examples.

The above cases show no settlement. The range is considerable, and, no doubt, in the case of the minimum pressure a much larger weight could have been imposed on the material without producing settlement. So that, for a safe rule, the average is low and a safe one would lie somewhere between the averages above given and the maximum pressures.

We find three cases where notable settlement took place in fine sand where the range was from 1·8 ton to 7·0 tons, and the average was 5·2 tons, no doubt the case of the minimum was one of loose quicksand unconstrained.

In Clay—largely cases of London clay—we find five examples where the pressures range from 4·50 tons to 5·60 tons, with an average of 5·2 tons, quite uniform pressures.

In silt and alluvium we have two cases of settlement which were 1·6 ton and 7·6 tons, a wide variation.

There are three cases of failure on sand and clay mixed, the pressures ranging from 1·6 ton (Chicago) to 7·4 tons, an average of 3·3 tons. It is to be noted that there was given above an average of 4·9 tons and ten examples, ranging from 2·5 tons to 8·5 tons, where no settlement occurred in similar material.

The records of frictional resistance are quite variable also. In ten cases of cylinder piers, the average was 540 lbs. per square foot, ranging from 300 lbs. to 1,500 lbs., gravel appearing to show the greatest amount (1,500 lbs.) and mud the least.

In respect to masonry piers, of which we have twenty-three examples, the range is from 300 lbs. per square foot in sand and gravel to 1,000 lbs. in sand and clay, with an average of 522 lbs. Walls, quays and otherwise, show an average of 270 lbs. per square foot, with a range from 205 lbs. to 450 lbs. with five examples.

The notes of Mr. Wychoff (see Appendix C) are of great value, as giving not only details of experience but the views of engineers who have either constructed the works or investigated them, or who have been called upon to report on the question of pressures and related subjects.

While many of the facts stated in these notes relate to subjects other than pressures, they nevertheless are necessary to clearly understand the principal features of the works, and also to give useful information to those seeking it in reference to such constructions.

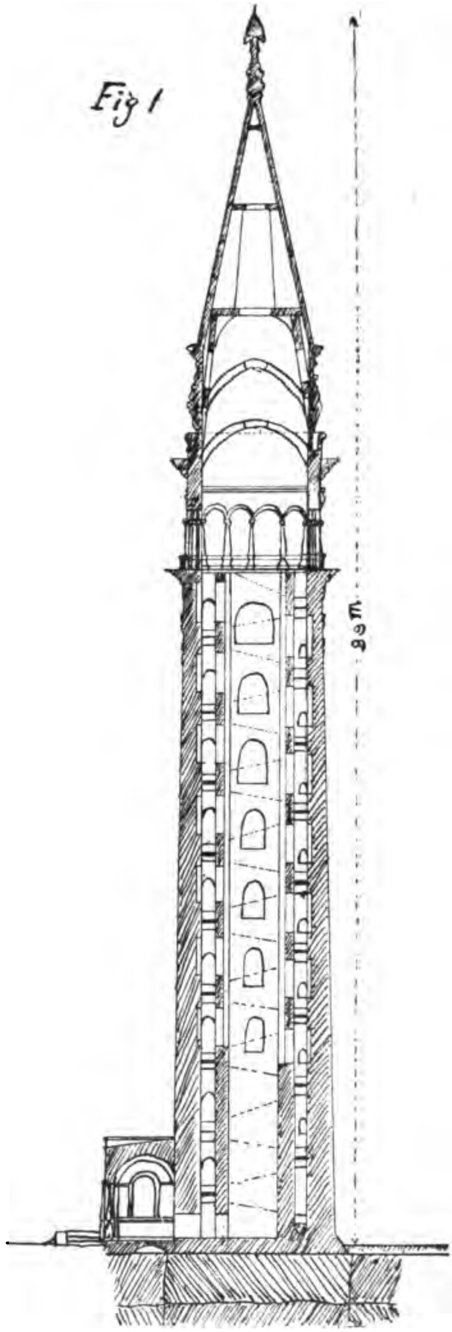
For this motive there are also in the notes many references simply to indicate where needed data may be found.

The writer will now give some interesting information from the sources that will be indicated.

In *La Ingenieria*, published at Buenos Ayres, March 31st, 1903, is correspondence from the Italian engineer Emilio Rosetti, of Milan, Italy, in reference to the fallen Campanile of San Marco, Venice. This letter to *La Ingenieria* was written immediately after it became known by telegraph that some mysterious accident had occurred with the construction of the new foundations of the Campanile. The soil upon and in which is built the City of Venice is composed of clay and sand strata, more or less resistant. The accompanying sketch shows the disposition of these strata as disclosed in 1885 by the examinations of the noted engineer, architect and archeologist, Boni, giving the entire situation, a vertical section, and plan of the Campanile, as well as the area about the old founda-

tions which it was intended to include in the new foundations (see Figs. 1 and 2). It was then discovered that the foundations did not cover as large an area as had been assumed, and covered only 222 square metres (263 square yards, or 2,368 square feet), the dimensions exceeding those of the square base of the Campanile by only 1 metre in each direction, this base having approximately 13 metres (42·6 feet) on each side. However, it was found that the foundations were in a nearly perfect state of preservation, and this condition was verified after the fall of the Campanile.

The section (see Fig. 2) shows that the surrounding material, after the filled ground had been passed through which was used to raise the square of San Marco, is composed of the following strata:—



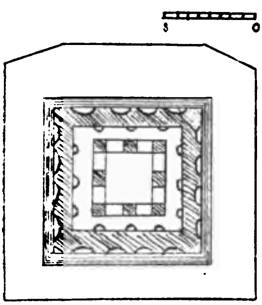
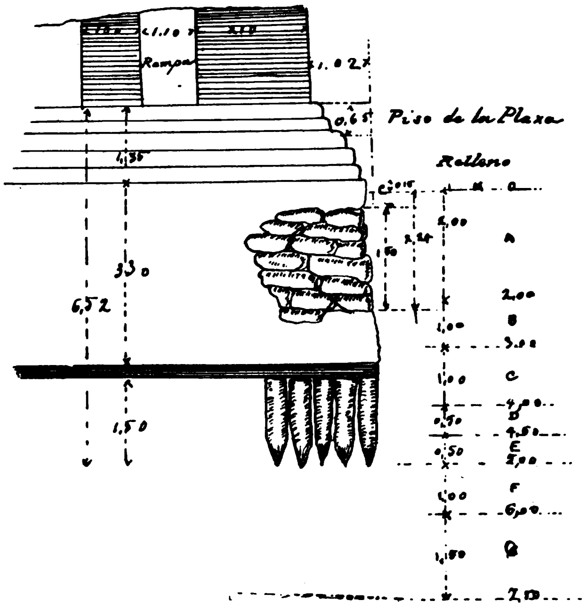


Fig 2.



	Thickness. Metres.	Feet.
A. Black clay and mud	2	6·56
B. Clay and mud	1	3·28
C. Compact clay with shells	1	3·28
D. Sandy clay	½	1·64
E. Sandy clay with shells	½	1·64
F. Sandy clay	1	3·28
G. Coarse clayey sand	1½	4·92
	7·5	24·60

Most of the monumental buildings of Venice reach with their foundations to stratum G (that is, about 3 metres (9·84 feet) below sea-level), on which is usually placed two courses of hard wood planks, generally oak, crossed at right angles. On this foundation is placed the masonry of rough stone or brick, which, with a slight batter, is carried up to the surface of the ground, where start the visible courses of the edifice. The foundations of the celebrated Doges Palace and the Sansovino Library were built in this manner. Sometimes, when it was not thought sufficient to rest the base of the masonry on planks laid on the material of stratum C or D, this plank base was placed on piles driven in the strata D and E, as was

done in the Basilica de San Marcos, and in the fallen Campanile (as shown in Fig. 2).

The piles were generally of elm (*Alnus Glutinosa*), a very resistant wood for the purpose; their diameter was about 0·25 metre (10 inches), and they were always very short, not generally more than 1·30 metre (4·3 feet) long, so as not to reach stratum F, and at any rate stratum G, which are permeable to the sea-water. The piles were driven very close together, nearly touching each other, in such a way as to compress the strata D and E and make them impermeable.

That this was a good foundation, regardless of its small area, is attested by the 1,000 years' existence of the Campanile. The very slight leaning of the tower was of little importance, and was evidently due to the unequal compressibility of the ground, or rather, to an unequal resistance in the foundations; this, however, being very small, did not contribute in any way towards its fall in 1902. The cause of this fall, as it is now known, was the careless and unskilful cutting away of the walls for purposes of admitting light and air; to give more room, and for other reasons, all of which weakened the shaft. A minute inspection and precise levellings referred to old bench marks show not the slightest subsidence of the foundations. Without going into the details of this examination, the facts about weight and pressure are relevant.

The weight of the Campanile was about 14,000,000 kilograms (31,742,000 lbs.) = 13,773 tons of 2,240 lbs., which gives on the 222 square metres of the base a mean pressure of 6·2 kilograms per square centimetre (5·6 long tons per square foot); adding pressure due to wind at 2·24 kilograms per square centimetre, we have as a maximum pressure 8·40 kilograms per square centimetre (7·7 long tons per square foot). This great pressure on such material would not be considered allowable in modern times—it would be "tempting Providence." Therefore two solutions were proposed: one to rebuild the tower on its historic lines, enlarging the foundations (as is shown in Fig. 1), reducing the normal pressure to 3·60 kilograms per square centimetre (3·29 long tons per square foot) and the maximum to 4·50 kilograms per square centimetre (4·1 long tons per square foot), which might possibly have been satisfactory, though it would have been difficult to enlarge the foundations and to connect them with the new. The other solution was to construct on the old foundations a lighter Campanile: the latter was not seriously considered, as the people demanded the old tower.

Mr. Beltrani, the Engineer and Architect, seeing that his opinions were not taken, resigned, and he was succeeded by the Architect

Moretti, who proceeded with the work of enlarging the foundations. This work went on for some time on the old method of driving piles around the old foundations, when suddenly a very grave occurrence suspended the work. It appears that several of the piles passed through the impermeable strata and reached the strata F and G (see Fig. 2), and the water penetrated, rising over the work and altering the condition of the adjacent material to such an extent that not only was there a liability of affecting seriously the old foundation but also those of surrounding buildings. What to do then was a serious question; probably the entire work will be removed, old and new, and new foundations provided. The future of this important work with its peculiar conditions will furnish useful information on the subject of pressures on foundations.

A very interesting Paper was presented to the Boston (Mass., U.S.A.) Society of Civil Engineers, January 28th, 1903, by Mr. Joseph R. Worcester, and was discussed by the members. The subject was "Boston Foundations." It was published in the June 1903 number of the Journal of the Association of Engineering Societies. Following is a brief synopsis of this Paper.

There is no provision in the Boston building laws—such as is often found in modern building laws—by which less than the full live load can be assumed to reach the bottom of the columns (evidently speaking of steel skeleton structures).

Bearing in mind the fact that it is scarcely possible to get the maximum live load over every foot of space of every floor, it seems we are amply justified to assume that not more than 50 per cent. of such maximum live load would ever come upon the foundations of the building.

The geologic formation of the "Boston Basis," so called, is something as follows:—

The underlying rock—a slate formation—is at a depth of from 50 to 170 feet below low water. This is overlaid with boulder clay in the form of smoothly rounded hills or drumlins, the thickness of which is from 15 to 90 feet. Above the boulder clay is found a layer of blue clay—a true glacial deposit, generally very tough and plastic, free from grit, but containing a large proportion of quartz flour with occasional thin layers and streaks of very fine sand with occasional angular fragments of rock.

This clay reaches to about 5 feet above high tide and was deposited in even horizontal layers. The solid rock below is, therefore, neither a help nor a hindrance to foundation work.

The structures generally rest upon two kinds of foundations, piles

or directly upon the soil. The piles are usually driven from 24 to 30 inches apart, centre to centre. In the higher parts of the city where the material is sand, or sand and gravel mixed, probably 10 tons per square foot could be placed upon it without appreciable settlement, no piles being used.

Architects generally assume 5 tons as a safe load, and this appears to be conservative. In the lower parts of the city the conditions are much different, and much depends upon the wetness of the soil. The softest clay encountered will carry about 2½ tons per square foot and up to 4 tons, and this accords with the experience of others where the clay is stiff, viz., from 2½ to 3½ tons, and where eccentricity of load has been considered, up to 3·85 tons, and, with pressure added for wind, 5·2 tons.

As stated by Mr. Edward S. Shaw in the discussion, who, in speaking of "frictional" resistance, prefers to call it "peripheral" resistance, to express the resistance to sinking caused by the adhesive pressure of earth upon the periphery of a pile, for the reason that this adhesion is not known to follow the well-known laws of friction given in the text-book on mechanics, he divides the total resistance into two components, viz., tip resistance and peripheral resistance, and considers it safe to neglect the former in all cases of soft ground, or where there is any uncertainty as to depth of the hard stratum to, or into, which the piles may be finally driven.

For the amount of peripheral resistance of piles, he considers values ranging from 200 to 500 lbs. per square foot safe for ordinary cases, the amount to be used depending upon the nature of the bottom, and its hardness, into which the piles are driven.

Some of the correspondence with which the writer has been favoured, through the kindness of his professional associates, is worthy of insertion in the appendix of this Paper. See Appendix C, where it is given entire, being communications from Mr. Corydon T. Purdy, of New York City; Mr. Joseph K. Freitag, of Boston, Mass., author of "Architectural Engineering;" and Mr. Bradford Leslie, of Harrow, England.

The general facts stated by them appear also in the tables accompanying this Paper.

APPENDIX B.

Pressures upon Four Datums.

(From Report of Messrs. Tullock and Noble on Tampico Wharf, being parts of Appendix B of that Report.)

Location.	Character of Material under Wall.	Maximum Pressure per Square Foot.	Upward Water Pressure.	Net Pressure on Earth.
Spezzia	Sand, silt and clay	6,700	2,250	4,450
Albert Dock . .	Sand	9,300	1,900	7,400
Hull	Sand	11,200	2,000	9,200
	Clay	11,400 14,700 13,800		11,400 14,700 13,800
Abercorn Basin, Belfast. . . .	Fine sand	12,100	900	11,200
Chatham Dock Yard Extension. . .	Gravel loam or chalk	10,800	2,100	8,700
Alexandra Dock, Belfast . . .	Sand overlying boulder clay	11,400	1,700	9,700
Alexandra Dock, Hull	Boulder clay or fine sand	17,400	2,500	14,900
Cork Quay . . .	Fine sand and gravel	13,000	1,400	11,600
Belfast Quay . .	Fine sand	11,400	1,800	9,600
South Dock, West India Docks . .	Clay (failed by sliding)	14,900	..	14,900
Avonmouth Dock .	Sand, clay (failed by sliding)	5,800	..	5,800

Friction on Cylinders and Piles.

(Being Appendix D of the Tampico Report.)

Friction on Cylinders.—Colson, Notes on Dock Construction, p. 387.

For cast-iron cylinders in gravel 1,050 to 1,400 lbs. per square foot for small depths and 1,400 to 1,700 lbs. at depths of 20 to 30 feet.

On brick and cement cylinders in the silt of the Clyde about 1,300 lbs. per square foot.

In sinking cylinder foundations of the Pragus-Smichow bridge by compressed air, skin friction was 314 lbs. per square foot.

Minutes Proceedings Inst. C.E., vol. xxii, p. 512.—Harrison Hayter gives observations on cast-iron cylinders of Charing Cross Bridge, sunk by divers, through 5½ feet of mud, 4 feet of sand and 23 feet of clay. Load required to sink 2,350 lbs. per square foot exposed to friction. Some of the load was probably required to overcome friction at the cutting edge.

Minutes Proceedings Inst. C.E., vol. cxxii, p. 187 *et seq.*—At Papaghni bridge, cast-iron cylinders 12 feet diameter and brick cylinders of same diameter. The following frictional resistances were determined while sinking these cylinders :—

	Cwts. per Square Foot.
In the upper sand	2·08 to 2·20
In black clay	3·50 ,, 5·60
In silt below clay	2·72 ,, 4·28
In the lower sand	2·58 ,, 3·16

Minutes Proceedings Inst. C.E., vol. ciii, p. 125 *et seq.*—The Chittrivatri bridge:—Cast-iron cylinders, 12 feet to 18 feet diameter, sunk open to hard material.

Friction through 33 feet sand, 10 feet clay, 7 feet clay and sand and clay and boulders—2·32 to 3·77 cwt. per square foot.

Friction through 33 feet sand, 10 feet clay and 3 feet sand and clay—2·93 to 3·62 cwt. per square foot.

Minutes Proceedings Inst. C.E., vol. ciii, p. 166.—Cylinder piers of bridges across the Tevy and Laira, 2 to 2·8 cwt. in mud.

Minutes Proceedings Inst. C.E., vol. l, p. 112 *et seq.*—Professor Jules Gaudard on Foundations.

Cast-iron cylinders sliding through gravel, 2 to 3 tons per square foot, for small depth and 4 to 5 tons at depth of 20 to 30 feet.

Minutes Proceedings Inst. C.E., vol. lxviii, p. 218.—River bridge. Brick coated with cement to reduce friction.

Weight required to sink—3 to 9½ cwt. per square foot. On cast-iron cylinders 2 to 7½ cwt. per square foot with fine sand, mud and coarse gravel.

Friction on Piles :—Minutes Proceedings Inst. C.E., vol. l, p. 112 *et seq.* In soft clay at La Rochelle and Rochefort, 164 lbs. per square foot.

In silt at Laurient, 123 lbs. per square foot. Colson, Notes on Dock Construction, p. 389.

In sand, firm, of good quality, the Dutch engineers estimate friction on piles at 614 lbs. per square foot.

Mr. Hurtzeg found, as the result of drawing some 300 piles, that the gross frictional resistance in clay was about 1862 lbs. per square foot.

APPENDIX C.—COMMUNICATIONS.

Letter from Mr. Corydon T. Purdy, dated New York, 25 March, 1905.

When we first began building high buildings in Chicago, and for several years afterward, we figured as high as 3,500 lbs. per square foot of "dead load" on the bottom of the footings, which, as you will remember, rest upon the soft blue clay which underlies the entire business portion of the city. This dead load included the weight of the building, with its foundations, but did not include any live, movable or super-imposed load. In the most conservative work of that period we rarely loaded less than 3,000 lbs.

The result proved to be wholly unsatisfactory. In all cases there was an immediate settlement extending through a period of years in some cases, including several of the most important buildings of Chicago ; this continued settlement has not yet disappeared, although it is now reduced to a very small fraction of an inch per year. In a few of the most important buildings the total settlement is now as much as 13 or 14 inches. The calculation of loads, however, was in all of these cases so well done that the settlement has been remarkably uniform, and the injury, therefore, is not nearly so great as might be inferred.

As a result of this experience, the present practice is entirely changed. Under the old plan the footings of a 16-or 18-story building would practically cover the entire available area. Now, footings spread out over the clay are not ordinarily used for buildings more than eight stories high. A maximum load of 2,800 lbs. to 3,000 lbs. is used, but a portion of the live or movable load is added to the dead load, sufficient to make the total load as nearly as possible equal to the actual one. Though the unit of load is not materially changed, the bearing resistance of the clay is greatly increased. This is due partially to the fact that the load taken now closely approximates the actual load, whereas originally it was only the weight of the materials of which the building was constructed. But the increase of resistance is due more to the fact that the area of the footings with a light building does not cover more than one-half or five-eighths of the total area of the lot.

Under these circumstances, the resisting areas of clay are really greater than the footings, and the settlement of such buildings is found to be immaterial.

Under higher buildings in Chicago, good practice now calls for pile foundations or concrete piers extending through the clay to solid rock or to a hardpan bottom. Where piles are used, they should in all cases be long, either long enough to reach to solid bottom, or very long and in sufficient quantity to be ample in their supporting power. Under the post-office and under the library piles were driven to hardpan. The use of concrete piers has steadily grown. Wells or open shafts can be dug for them through the clay very easily without compressed air. The

clay moves and compresses, but it does both very slowly, and it practically excludes the water. Advantage is taken of these conditions. A section of the well is dug and lined up as before with the wooden staves, and the process repeated again and again until the bottom is reached. These concrete piers extend down ordinarily about 80 feet below the level of the street. The rock throughout the business section of the city is covered by a hardpan of clay and sand and gravel, mixed in different proportions. It is very hard but does not retain the water as the sand and clay which overlies it. If the concrete pier is pushed through to the rock, in places it is liable to involve great cost on account of the water pressure in the hardpan. Ordinarily the concrete piers are loaded about 4,500 lbs. at the top of the concrete. If the piers go through to the rock they are kept practically the same dimensions all the way down. In many cases, however, it seems advisable to stop them on the top of the hardpan (which is a water-bearing strata composed of gravel, shale, sand and clay), in which case the diameter at the bottom is made double that at the top, or, if square, enlarged proportionately. In same cases the hardpan has considerable depth, and in others there is very little of it. If there is little of it, it is better to go to the rock ; if there is a good deal of it, it is usually more economical to stop on top of the hardpan.

In New York practice varies widely, both on account of a great diversity of conditions and also on account of the large number of different designers. In Chicago the designing of large buildings is practically in the hands of a few men. In New York every architect aspires to that class of work, and likewise many engineers are employed. In the lower part of the town, where, from the water line to the rock, the ground formation is a silt of the very worst kind, the best and largest buildings are carried on concrete piers put in under compressed air. The Lord's Court and the Park Row buildings are on piles. The North American Trust, American Exchange Bank, St. Paul and the Broadway Chambers are notable examples of buildings built on the sand, much in the same way as the first big buildings in Chicago were founded on the clay, except the loads are greater.

Under the New York building law the amount of live or movable load carried by the columns is reduced in accordance with a given formula. It has been our practice to use the whole load for our footings, that is, a load including the entire weight of the building, and a fraction of the live load, the same as required for the basementc olumns. When the underlying sand strata is good we have loaded to four (4) tons per square foot, with no material settlement, the full limit which the law allows. In cases where the sand is not good this amount is somewhat reduced.

In many of the most important uptown buildings the foundations, as you know, can be easily carried on the rock, but the exact details even of this simpler work vary considerably with different buildings, and the amount carried by the footing varies with changing conditions.

I hope this memorandum with reference to our practice on footings will prove to be of some service to you. The statement has been carefully prepared, and I think all that is in it can be relied upon as representing the best practice in these particulars.

Very truly yours,

(Signed) CORYDON T. PURDY.

Extract from Letter of Mr. Jos. K. Freitag, dated Boston, Mass., June 12th, 1902.

Regarding foundation loads under pneumatic caissons, etc., I fear that I would be able to give Mr. Corthell but little information. The examples which I quote in my "Architectural Engineering" are necessarily entirely confined to building practice, as I do not go outside of high buildings in any way. If building practice is at all interesting to you I might state that the pneumatic caissons in the Manhattan Life Building in New York city resulted in a pressure per square foot at base of caissons estimated at 10·8 tons per square foot, those for the Gillender, an extremely narrow twenty-story building, resulting in an estimated pressure of 12 tons per square foot, while the pressure per square foot under the American Surety Building, also pneumatic, was estimated at about 14,500 lbs. These, I believe, were all on rock-bed or hardpan.

For building foundations on sand the New York World Building is built upon inverted arches upon continuous concrete footings, the foundation material being a dense fine sand, the resulting load being 4·7 tons per square foot. The St. Paul Building (also in New York) is built upon extremely compact sand, the foundations being a continuous grillage over the entire lot, the resultant pressure being 3·2 tons per square foot. The Spreckles Building (S. Francisco), also built with continuous grillage over dense wet sand, used a unit pressure of 4·500 lbs. per square foot.

The units used for Chicago buildings you are probably familiar with, these being on the upper stratum of hard clay. As you probably know, the best results have been obtained from the use of 3,000 to 3,500 lbs. per square foot. Examples which I know of ranged from 2,850 to 3,750 lbs.

Prof. Baker, in his "Masonry Construction," quotes a number of interesting examples on foundation pressures, and if you do not happen to have a copy of Baker at hand I enclose herewith a few notes which might possibly be interesting. I also enclose a few notes from Mr. Morison's report on the Plattsmouth Bridge which will give you the unit pressures which he used in that instance.

Notes on Foundations, from Baker's Treatise on Masonry Construction.

§ 276. The following data on the bearing power of clay will be of assistance in deciding upon the load that may safely be imposed upon any particular clayey soil. From the experiments made in connection with the construction of the capitol at Albany, New York, as described in § 271, the conclusion was drawn that the extreme supporting power of that soil was less than 6 tons per square foot, and that the load which might safely be imposed upon it was 2 tons per square foot. "The soil was blue clay containing from 60 to 90 per cent. of alumina the remainder being fine siliceous sand. The soil contains 27 to 43, usually about 40, per cent. of water, and various samples of it weighed from 81 to 101 lbs. per cubic foot." In the case of the Congressional Library (§ 271) the ultimate supporting power of "yellow clay mixed with sand" was 13½ tons per square foot, and the safe load was assumed to be 2½ tons per square foot. Experiments made on the clay under the pier of the bridge across the Missouri at Bismarck, with surfaces 1½ inch square, gave an average ultimate bearing power of 15 tons per square foot.

The stiffer varieties of what is ordinarily called clay, when kept dry, will safely bear from 5 to 6 tons per square foot; but the same clay, if allowed to become saturated with water, cannot be trusted to bear more than 2 tons per square foot. At Chicago the load ordinarily put on a thin layer of clay (hard above and soft

below, resting on a quick stratum of quicksand) is 1½ to 2 tons per square foot, and the settlement, which usually reaches a maximum in a year, is about 1 inch per ton of load. Experience in Central Illinois shows that, if the foundation is carried down below the action of frost, the clay subsoil will bear 1½ to 2 tons per square foot without appreciable settlement. Rankine gives the safe load for compressible soils as 1¼ to 1⅔ tons per square foot.

§ 278. Compact gravel or clean sand, in beds of considerable thickness protected from being carried away by water, may be loaded with from 8 to 10 tons per square foot with safety. In an experiment in France, clean river sand compacted in a trench supported 100 tons per square foot. Sand well cemented with clay and compacted, if protected with water, will safely carry 4 to 6 tons per square foot.

The piers of the Cincinnati Suspension Bridge are founded on a bed of coarse gravel 12 feet below low water, although solid limestone was only 12 feet deeper; if the friction on the sides of the pier be disregarded, the maximum pressure on the gravel is 4 tons per square foot. The piers of the Brooklyn Suspension Bridge are founded 44 feet below the bed of the river upon a layer of sand 2 feet thick resting upon bed-rock; the maximum pressure is about 5½ tons per square foot.

At Chicago sand and gravel about 15 feet below the surface are successfully loaded with 2 to 2½ tons per square foot. At Berlin the safe load for sandy soil is generally taken at 2 to 2½ tons per square foot. The Washington Monument, Washington, D.C., rests upon a bed of very fine sand 2 feet thick underlying a bed of gravel and boulders; the ordinary pressure on certain parts of the foundation is not far from 11 tons per square foot, which the wind may increase to nearly 14 tons per square foot.

§ 280. *Semi-liquid Soils.*— It is difficult to give results of the safe bearing power of soils of this class. A considerable part of the supporting power is derived from the friction on the vertical sides of the foundation, hence the bearing power depends in part upon the area of the side surface in contact with the soil. Furthermore, it is difficult to determine the exact supporting power of a plastic soil, since a considerable settlement is certain to take place with the lapse of time. The experience at New Orleans with alluvial soil and a few experiments that have been made on quicksand seem to indicate that with a load of ½ to 1 ton per square foot the settlement will not be excessive.

§ 281. *Bearing power. Summary.*—Gathering together the results of the preceding discussion, we have the following table:—

SAFE-BEARING POWER OF SOILS.

Kinds of Materials.	Safe-bearing power in tons per square foot. Minimum.	Maximum.
Rock, the hardest, in thick layers in native bed, 274 .	200	..
„ equal to best ashlar masonry	25	30
„ „ „ brick „	15	20
„ „ „ poor brick masonry	5	10
Clay, in thick beds, always dry, $276	4	6
„ „ „ moderately dry	2	4
„ soft, $276	1	2
Gravel and coarse sand, well cemented	8	10
Sand compact and well cemented, $278	4	6
Sand, clean dry	2	4
Quicksand alluvial soil soils	0·05	1

§ 455. *Frictional resistance.*—At the Havre de Grace Bridge the normal frictional resistance on the timber sides of the pneumatic caisson was 280 to 350 lbs. per square foot for depths of from 40 to 80 feet, the soil being silt, sand and mud. When boulders were encountered, the resistance was greater, and when the air escaped in large quantities the resistance was less. At the bridge over the Missouri River near Blair, Nb., the frictional resistance usually ranged between 350 and 450 lbs. per square foot, the soil being mostly fine sand with some coarse sand and gravel and a little clay. At the Brooklyn Bridge the frictional resistance at times was 600 lbs. per square foot. At Cairo, in sand and gravel, the normal friction was about 600 lbs. per square foot.

COEFFICIENT OF FRICTION OF MATERIALS AND SURFACES USED IN FOUNDATIONS.

Kinds of Materials.	For Dry Materials.		For Wet Material.	
	At beginning of Motion.	During Motion.	At beginning of Motion.	During Motion.
Sheet iron without rivets on gravel and sand	0·40	0·46	0·33	0·44
Sheet iron with rivets on gravel and sand	0·40	0·49	0·47	0·55
Cast iron (unplaned) on gravel and sand	0·37	0·47	0·36	0·50
Granite (roughly worked) on gravel and sand	0·43	0·54	0·41	0·48
Pine (sawed) on gravel and sand	0·41	0·51	0·41	0·50
Sheet iron without rivets on sand	0·54	0·63	0·37	0·32
,, ,, with ,, ,, ,,	0·73	0·84	0·52	0·50
Cast iron (unplaned) on sand	0·56	0·61	0·47	0·58
Granite (roughly worked) on sand	0·65	0·70	0·47	0·53
Pine (sawed) on sand	0·66	0·73	0·58	0·48

FOLLOWING FROM GEO. S. MORISON'S REPORT ON THE PLATTSMOUTH BRIDGE.

The pressure on the foundations of the principal pieces of masonry is as follows:—

Pier I.—At River Bank.

	Lbs.
Concrete (25,000 cubic feet at 52½ lbs.)	1,312,500
Masonry below 497 (37 cubic yards at 2,565 lbs.)	94,905
Masonry above 497 (825 ,, ,, ,, 4,250 ,,)	3,506,250
Superstructure	516,000
Estimated moving load	500,000
Total	5,929,655

Area of base	1,000 square feet
Average pressure per square foot	5,900 lbs.
,, ,, ,, ,, inch	41·2 lbs.

Pier II.—In River Bed.

	Lbs.
Caisson and filling (16,600 cubic feet at 34¼ lbs.) . .	568,550
Crib work and filling (14,850 cubic feet at 38½ lbs.) .	571,725
Masonry below 497 (40 cubic yards at 2,565 lbs.) . .	102,600
Masonry above 497 (824 yards at 4,250 lbs.). . . .	3,502,000
Superstructure	1,000,000
Estimated moving load	850,000
Total	6,594,875

Area of base	1,071 square feet
Average pressure per square foot . .	6,158 lbs.
" " " " inch . .	42·8 lbs.

Pier III.—At Bank, Partly in Stream.

	Lbs.
Caisson and filling (16,600 cubic feet at 34¼ lbs.) . .	568,550
Concrete and crib work (29,450 cubic feet at 38½ lbs.)	1,133,825
Masonry below 497 (110 cubic yards at 2,865 lbs.) . .	282,150
Masonry above 497 (830 cubic yards at 4,250 lbs.) . .	3,532,500
Superstructure	1,680,000
Estimated moving load	650,000
Total	7,847,025

Area of base	1,071 square feet
Average pressure per square foot . .	6,393 lbs.
" " " " inch . .	44·4 lbs.

Pier IV.—On the Flats.

	Lbs.
Caisson and filling (11,160 cubic feet at 25 lbs.). . .	279,000
Concrete and crib work (24,500 cubic feet at 36 lbs.) .	882,000
Masonry (258 cubic yards at 4,250 lbs.)	1,096,500
Superstructure	360,000
Estimated moving load	450,000
Total	3,067,500

Area of base	750 square feet
Average pressure per square foot . . .	4,090 lbs.
" " " " inch . .	28·3 lbs.

Pier V.—On the Flats.

	Lbs.
Grillage and concrete (2,000 cubic feet at 102 lbs.) .	204,000
Masonry (259 cubic yards at 4,250 lbs.)	1,100,750
Superstructure	360,000
Estimated moving load.	450,000
Total	2,114,750

Number of piles	78.
Average weight per pile.	27,112 lbs.

Pier VI.—On the Flats.

	Lbs.
Concrete (1,188 cubic feet at 115 lbs.)	136,620
Masonry (148 cubic yards at 4,250 lbs.)	629,000
Superstructure	190,000
Estimated moving load	275,000
Total	1,230,620

Area of base	386 square feet
Average pressure per square foot	3,107 lbs.
„ „ „ „ inch	21·6 lbs.

Small Piers under Viaducts.—On the Flats.

	Lbs.
Concrete (50 cubic feet at 115 lbs.)	5,750
Masonry (90 cubic feet at 150 lbs.)	13,500
Superstructure	15,000
Estimated moving load	50,000
Total	84,250

Area of base	25 square feet
Average pressure per square foot	3,370 lbs.
„ „ „ „ inch	23·4 lbs.

In estimating the pressure on the foundations of piers I, II, III and IV, a deduction has been made for the water displaced by the immersed portion of the piers at low water. To obtain the actual pressure on the foundations, this deduction should not be made; but to get the relative pressure, that is, the increased pressure on the foundation over and above that on the surrounding surface, which is the real measure of the labour of the foundation, this deduction should be made. The actual pressure is of course equal to the whole weight of the material in the pier with the addition of the atmospheric pressure.

Letter from Sir Bradford Leslie, Harrow, England, dated April 15th, 1903.

Referring to Dr. Corthell's Circular of the 5th December, 1902, received through the Secretary of the Institution of Civil Engineers, Westminster, I regret that pressure of business and loss of records forbid any attempt on my part to reply systematically to the twenty-one heads under which information is requested.

The pressure on foundations of bridge piers is referred to in my description of the bridge over the Gorai River (Minutes of Proceedings Inst. C.E., 1872, pp. 42, 43, 44). Plate III. of my Paper on "Bridges in the Bengal Presidency," published by the Royal Engineers Institute, Chatham, gives a section of the Gorai River, showing the bridge in 1889. The section shows by the upper line the river bed as scoured out in the flood season. The mounds round piers Nos. 3, 4 and 5 show the rubble stone deposited to limit scour. When first sunk and before it was

protected by stone rubble, the scour reached within 18 feet of the bottom of No. 5 pier, consequently it could have received but little support from lateral friction, and practically the full 8½ tons (English) per square foot was transmitted to the base area of the pier, which rested upon a stratum of clean grey sand. The base of pier No. 5 is about 90 feet below water level in the dry season. The sand being permeable, the pier is water-borne to the extent of the water pressure at that depth, say 2½ tons per square foot. This deducted from the total pressure of 8½ tons leaves 6 tons as the maximum pressure on the sand foundation. In the flood season the pier is, of course, water-borne to an increased extent, proportionate to the increased depth of water.

In my Paper on the Jubilee Bridge (Minutes of Proceedings Inst. C.E., 1888, p. 55) some remarks on lateral pressure or skin friction will be found (see Tables —Author).

On pp. 31, 32 and 33 of my Paper on "Bridges in the Bengal Presidency," referred to above, observations on the weight in the bases of piers and on skin friction occur. On p. 31 skin friction is given as 1 cwt. on the total area of pier embedded for every 20 feet in depth. Thus a well or cylinder of 20 feet diameter embedded 40 feet would have a total area embedded of 62·8 feet × 40 = 2,512 square feet × 2 cwt. = 251 tons (English) effective skin friction. This is during the operation of sinking. The difficulty of re-starting a cylinder or well, if sinking is suspended for some time, indicates that skin friction increases with repose, and for statical purposes it would probably be safe to take 1 cwt. on the total area embedded for every 10 feet in depth as the support that may be relied for skin friction. This is not in accordance with the view expressed at the bottom of p. 43 of the Paper on the Gorai River, but it must be remembered that at the date of that Paper (1872) there was very little experience on this subject.

Almost all railway bridges over large rivers in India are carried by brick piers supported on single or multiple wells or cylinders, or on caissons sunk to a safe depth into the river bed and hearted with concrete, scour being limited by depositing rubble stone round the wells or caissons. As will be seen by the Paper on "Bridges in the Bengal Presidency," many instances have occurred of such piers being undermined and thrown over by scour produced by eddies in the flood season, but I do not know of a single instance of vertical settlement of such piers.

By the Paper on the "Hooghly Floating Bridge" (see Tables—Author) (Minutes of Proceedings Inst. C.E., 1877) it will be seen, p. 7, that the abutments are founded at no great depth, on river silt, practically quicksand, so soft that the iron bowsprit of a ship (broken off in the cyclone of 1864) had sunk down in the silt and had to be extracted from the site of the abutment at foundation level. The bridge rises and falls with the tide, and, being hinged to the abutments, any settlements by dislocating the hinge axis would have been disastrous. None, however, has occurred, In 1859 I founded a 20-foot brick arch for passing the Mooktapore Khall (a tidal creek) under the Eastern Bengal Railway on a quicksand so soft that a man could not cross it. The tide was kept out of the creek by a dam, but we only got down to the foundation level by constant pumping; fountains of silt and water were sprouting up over the whole area of the foundation, which was so liquid that it found its own level. Jungle-wood planks were laid over the whole area of the quicksand to carry the first thin layer of concrete, each subsequent layer increasing in thickness. An inverted arch of brickwork and the abutments were built on the concrete. The centre was then erected and the arch turned, and when the centring was struck there was not a crack in the structure. This bridge has carried the heavy traffic of the Eastern Bengal

Railway ever since. Of course I built this small arch on a silt quicksand by way of experiment. It was the success of this experiment that, many years afterwards, made me feel safe in founding the abutments of the Hooghly Floating Bridge on quicksand. Provided there is no risk of scour and the concrete platform is got in below the minimum height of saturation (this is important), and is made of a thickness adequate to distribute concentrated weights over a sufficient area, a running quicksand is an efficient foundation. Any lowering of the level of saturation or drainage of the quicksand would however be fatal to the stability of the structure.

During 29 years of constant employment in bridge building in India I made many observations and collected notes on these matters. Unfortunately I lost all my memoranda in the wreck of the S.S. "Tasmania" when I was returning from India, 1887; if, however, the foregoing observations should be of any service to Dr. Corthell, I should be much gratified.

Yours truly,

(Signed) BRADFORD LESLIE.

APPENDIX D.

PRESSURES ON DEEP FOUNDATIONS.

By Elmer Lawrence Corthell, D.Sc., M. Inst. C.E.

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APPENDIX D.

The Foundations of the New Croton Dam. By CHAS. S. GOWEN. Trans. Am. Soc. C.E. Vol. 43, p. 469.

In 1883 the Legislature of the State of New York passed an Act creating the Aqueduct Commissioners of the City of New York.

The purpose of this Act was the immediate increase of the water supply of the city which, under the conditions then prevailing, had for some time been inadequate and insufficient. To this end it was planned to begin the construction of a new aqueduct and a large dam on the Croton River, the latter near to and above the site of the Quaker Bridge at a point about 4 miles below the old Croton Dam, which had been in use since 1839. This new dam, it was reckoned, would increase the available storage by about 32,000 million gallons. In January 1891 the Commissioners decided to build the large dam at the Cornell site, a point about 1¼ miles above Quaker Bridge, and so situated as to store nearly as much water as would have been stored by the Quaker Bridge Dam, 30,000 million gallons.

The new Croton Dam at Cornell site, which is to form the longest reservoir of the system, on the lower part of the Croton River, was begun in October 1892.

It is located about 3½ miles above the junction of the Croton with the Hudson, and about 1 mile above the Old Quaker Bridge. The course of the Croton at this point is approximately west.

At the dam location rock (gneiss) crops out on the surface on the north side of the river, rising with a steep slope, which terminates at the top of a hill about 300 feet high. The bedrock on the north side dips quickly just before reaching the bank, and soundings show it at about 75 feet below the river bed. At this point, on a line about parallel to and under the river, the rock changes abruptly from gneiss to limestone, extends across the valley at about the depth noted above, with some deeper pockets, and then rises gradually on the south side with the earth slope and below it, at varying depths, to a depth of about 20 feet at the extreme end of the dam location.

Under the river bed the material above bedrock is largely sand, gravel and boulders. Approaching the south side of the river valley, very compact hard pan and gravel next to the rock is indicated. The hardpan is surmounted next to the surface by a considerable layer of sand at the steep part of the slope ; at about elevation runs the Old Quaker Aqueduct. The total distance across the valley at flow line (elevation 200) is about 1,300 feet.

The masonry dam will be about 710 feet in length from its junction with the overflow to the back of the wing-wall at the south end, and its extreme height will be 260 feet or more, as the soundings show some large and deep depressions on the rock surface below. Maximum thickness at bottom next to rock surface below. Maximum thickness at bottom next to rock about 190 feet.

The dam was designed by Mr. Fteley and assisted by Mr. Wegmann. (Safety factor of 2 against any tendency to overturn.)

Owing to the character of the limestone, which rendered deep excavation necessary at certain points, the extreme height of the masonry dam will range from elevation 80. The lowest point reached at the foundation excavating to elevation 210, a total of 290 feet. For the same reason the extreme thickness of the main dam masonry|at the toe is 200 feet. P. 487. Earth Excavation. Main Dam foundation.

This work involved preparation for a foundation on rock extending from about Sta. 3 + 30 to about Sta. 10 + 00, where the new river channel, formed in connection with the protection work, is merged into the foundation, and which varies in width from about 200 feet at the lowest point to about 130 feet at Sta. 10 + 00 and 140 feet at Sta. 3 + 30 on the line of the back of tho proposed wing wall. The necessary earth excavated covering this area was about 885,000 cubic yards, consisting largely of loose sand, gravel and boulders, with, however, at the south end of the pit, a large area of hardpan excavated, this hardpan forming to a considerable extent the slopes at this end of the excavation.

Rock excavation and foundation for main dam, p. 494.

The rock on the north side of the valley, on the steep side hill, cropped out at points very near the surface. It was formed of gneiss, considerably fissured, but generally sound after reaching a certain depth in the ledge. This gneiss extended to the line of the old bed of the river, when its depth below the surface was much greater, being about 75 feet.

P. 497.

In limiting the extent of the excavation vertically, the end aimed at was to reach rock sufficiently free from seams and solid enough to afford all the bearing strength necessary to sustain the superimposed masonry and resulting pressure. The result involved a very large amount of deep rock excavation, the depth in one place being 50 feet, before satisfactory compact rock was found.

(Professor Kemp made report concerning caves in limestone rock.) Caves not frequent. Several however found. P. 502. Between Sta. 7 + 30 and 7 + 60 is shown a narrow, well-defined seam of hard rock, with many erosions connected and extending to the deeper holes excavated at the ends. Beyond this seam lies a compact seam of friable limestone about 10 feet wide. It was tested for bearing strength by an apparatus shown in *Fig. 10.* This apparatus consists of a cylinder to be loaded with shot necessary to produce the required pressure upon its bearing point, a circle ¼ inch diameter. This was applied carefully and repeatedly to the surface in question at different points, and the results indicated that the bearing power of the surface was ample up to the limit of the test, which was 250 lbs. to the square inch.

P. 560. Pressures (calculated) were limited to 15 tons per square foot at the base of the structure (rock surface) and the lines of pressure were kept well within the middle ⅓ of the section at any assumed level.

Some Observations on the Deep Pneumatic work of the Resident Engineer, New East River Bridge Foundations. By EDWIN DURYEA, Jun. "Engineering News," 1st May, 1902, p. 358.

The highest pressure ever used in North Brooklyn Pier caisson was 38 lbs. per square inch, and this only when the cutting edge first reached the depth of 90 feet below mean high water. The pressure was then at once reduced to 31 lbs. per square inch for a couple of days, then carried for the remainder of the pneumatic work (a period of over 6 weeks) at pressures of 33 lbs. to 37 lbs. per square inch, the variations being irregular and the pressures not increasing with the depth.

The preliminary diamond-drill borings showed above the clay about 50 feet of water and about 20 feet of sand, with some boulders. The clay began at about 70 feet below mean high water and extended without change to the rock, the highest point of which was found at a depth of 105 feet. The depths found by the diamond drill were in general corroborated by the more exact information secured during the sinking. This latter information showed that unless borings are taken very close to each other no inference is warranted as to the shape of the bedrock at intermediate points, at least with the gneiss rock of this neighbourhood. The diamond-drill borings and the sinking also showed that no reliance could be placed on wash borings to distinguish between boulders and bedrock.

With the diamond-drill borings as data, the caisson, masonry and coffer dam of the north pier[1] were designed of such heights as corresponded to a final depth of the cutting edge of 100 feet below mean high water or 5 feet above the deepest known rock. The clay was of such hardness and of so great a depth that entirely suitable foundations for any bridge, except perhaps a suspension bridge, would have been secured by sinking the cutting edge only a few feet into it and not trying to reach the rock. The slightest settlement would affect the verticality of the high tower, however, and in this case the rock could be reached by the pneumatic process at what was proportionally only a very small addition to the cost of the bridge; the only wise and conservative treatment was to carry the foundations to rock. In case the rock had been much deeper, it is very probable the clay would have been used as a foundation. It was fully equal for the purpose to the clays on which are founded the truss bridges over the Missouri River at Rulo and Bismarck, and the long span cantilever bridge over the Mississippi River at Memphis.

When the caisson of the north pier was landed on the river bottom its cutting edge was about 49 feet below mean high water. The ordinary fall of the tide was about 5 feet; the river bottom was covered by about a foot of very offensive sewer mud, and below this, to a depth of 63 to 72 feet (average about 68½ feet), extended a bed of sand, gravel and cobbles, with some boulders. Below this lay a bed of hard, dry, stratified clay extending to the rock, the highest point of which was found at a depth of about 84½ feet and the lowest at 107·5 feet. The average depth of the original rock surface within the whole area at the caisson was 96·5 feet. *The maximum depth of 110 feet, which has been published several times, is incorrect.*

The cutting edge was stopped at a depth of 95 feet (5 feet higher than was originally intended), 46 per cent. of it being then within a few inches of the excavated rock surface, and the remainder an average of 7 feet and an extreme height of 11 feet above the original rock. The excavated rock surface covered 42 per cent. of the area of the caisson. The mean depth of the final rock surface over the whole area of the caisson is 98·3 feet below mean high water.

The comparatively small amount of work done at the greatest depth is shown by the amount of excavation being only about 2 cubic yards below the depth of 106 feet, 21 cubic yards below 104 feet, 77 cubic yards below 102 feet, and 183 cubic yards below 100 feet. The size of the caisson was 63 feet by 79 feet, giving a volume per foot depth of its whole area of 184 cubic yards. The deeper parts were left uncovered as short a time as possible, about 40 cubic yards of excavation being done after the concreting was begun. The extreme depth of 107·5 feet,

[1] "Engineering News," 27th May, 1897.

existed for less than an hour, concrete being placed on the rock within a few minutes after it was satisfactorily cleaned.

The indicated air pressure exceeded that corresponding to the depth of cutting edge by several pounds (generally from 3 to 6 lbs.) from the beginning down to a depth of about 75 feet, or until all parts of the edge were well within the clay. From this depth to about 90 feet the two pressures corresponded quite closely, sometimes one and sometimes the other being slightly larger. Below 90 feet, as already mentioned, the indicated pressure was much smaller than that corresponding to the depth of water.

When the excavation is carried 11 feet vertically below the cutting edge, however, to a depth of 106 feet below mean high water, and the pressure maintained is not more than 36 lbs. per square inch, some degree of danger may be thought to have been incurred. The depth corresponding to a pressure of 36 lbs. is 81 feet, or 14 feet less than the depth of the edge and 25 feet less than the extreme depth of the vertical clay face. The pressure corresponding to the depth of cutting edge is 42·2 lbs., and that corresponding to the extreme depth of 106 feet is 47 lbs.

From information furnished by the borings it was decided that it would be conservative to take advantage of the bed of clay to stop the cutting edge at a height of at least 5 feet above the deepest rock. As actually built the edge was stopped 5 feet higher than intended, or at 95 instead of 100 feet depth.

For the south end of the caisson the cutting edge has between it and the rock only a few inches of concrete, while for the north half it has an average intervening depth of 7 feet and a maximum of 11 feet. This concrete will have an excess pressure from the completed bridge of only about 5 tons per square foot, while it might safely in this location bear 15 tons. The size of the pier was governed by the lateral dimensions of the tower leg instead of by unit working pressures.

The clay was very peculiar in its appearance and quite unusual. It was very uniform and markedly stratified, resembling at a few feet distance a strongly laminated gneiss. It was in fact supposed to be a decomposed gneiss rock until some small boulders were encountered in it. Its junction with the bed rock was plainly marked, only an inch or two of harder material separating them. The clay was free from sand, quite dry, and would absorb very little water, though by continual working it would become greasy and putty-like. It was so hard that it was found economical to excavate a great deal of it by blasting with dynamite, though it was not impracticable to excavate it by pick and bar. Vertical faces were soon affected by exposure to the air, pieces varying in size from a pail to a barrel detaching themselves slowly. This was easily prevented, where necessary, by planks.

The quality of the clay, its soundness—the stratification showing that it had not been disturbed since its original deposition—and its great thickness would have justified risks much greater than any taken. The minimum thickness of clay above the edge was 23 feet, and this was overlaid by a minimum thickness of 36 feet of sand and excavated materials. No water was ever found leaking down from between the outside surface of the caisson and the clay.

The piers were built by the Degnon-McLean Construction Company, and the success attending the difficult pneumatic work was due principally to the good judgment and untiring watchfulness of J. E. Taber, their foreman in general charge of the sinking.

The St. Louis Bridge from Observations on Deep Pneumatic work of the New East River Bridge Foundation. By EDWIN DURYEA, Jun. "Engineering News," 1902, p. 360.

The most familiar example of high pressure is the St. Louis Bridge, where the sinking was through sand to rock. The east pier, when landed on rock, had its cutting edge immersed 93·2 feet, corresponding to over 40 lbs. per square inch. The last 34 feet was sunk in 27 days, or at an average rate of 15 inches per day. Filling of the chamber with concrete was begun 7 days later and discontinued after 38 days, when the immersion had reached a depth of 111·75 feet. The concreting was resumed after an intermission of 28 days, the river having fallen to an immersion of 107½ feet, and was completed 16 days later. The concreting of chamber lasted 53 working days (in two disconnected periods) and was done in pressures of 45 lbs. to the square inch and 50 lbs. to the square inch, corresponding to theoretical immersions of about 104 to 115 feet.

The only other deep sinking at the St. Louis Bridge was the sinking of the east abutment. When its cutting edge was 14 feet 9 inches above the rock, with an immersion of 97·3 feet, the pressure carried was 43 lbs.; 9 days later, with its edge 10·1 feet above rock, an immersion of 101·1 feet and a pressure of 46 lbs., the work was stopped by a tornado.

When landed on the rock the edge was immersed 109·7 feet and the pressure was 49 lbs., 2 days later the placing of concrete under the edge and girders was begun and completed in 17 days. The remainder of the chamber was then pumped full of sand and water.

Memphis Bridge. By EDWIN DURYEA, "Engineering News," 1902, p. 360.

The most notable and successful instance among bridge foundations of long protracted continuous work under heavy pressure is the sinking of the 2-inch river piers of the Memphis Bridge, though but little attention seems to have been drawn to it. This work was done by days' labour under the direction of Mr. Alfred Noble as Resident Engineer. The piers are on clay foundations and were sunk through sand and a few feet into the clay. In pier No. 2 the immersion of edge was for 11 days from 100 to 104 feet, with indicated pressure of 41 lbs. to 43 lbs. and calculated pressures of 43 to 45 lbs. At a later period the immersion was from 93 to 104 feet for 50 days, the indicated pressure being 40½ to 44½ lbs. In pier No. 3 the immersion was for 21 days from 90 to 105 feet with indicated pressures of 37 to 44½ lbs. and calculated pressures of 39 to 45½ lbs. For 75 days at a later date the immersion was 94 to 106·4 feet with the indicated and calculated pressures respectively 40 lbs. and 41 to 46 lbs.

The Foundations of the St. Lawrence Bridge. BY G. H. MASSY, Canadian Society of C. E. Trans., Jan. 2, 1887–89, p. 36.

In the autumn of 1881 the Atlantic and North Western Railway Co. decided to build a bridge across the St. Lawrence in the vicinity of Montreal. The soundings at the point where the bridge now stands showed the existence of an irregular reef about 500 feet wide extending from the north shore to the main channel, with a depth of from 5 feet to 20 feet of water. The current here runs at a

speed of from 2½ miles to 6 miles per hour at low water, and from 4 miles to 9 miles at high water, the difference between high and low water being about 6 feet.

The borings showed bare rock near the north shore, but towards the centre the bottom was covered to a depth of several feet with gravel and hard pan. The rock forming the bottom of the river is mostly Utica shale interspersed with veins and floors of trap. Above this formation the blue limestone appears on the south shore.

The foundation (for pier No. 4) here was bare rock, so that all that was required to be done was to get the caisson into place and commence concreting. The caisson was built of 12-inch by 12-inch timber.

No. 13 pier was always looked upon as the most difficult. It stands in 28 feet of water and at the swiftest part of the current, and on it are to rest the cantilever spans of 408 feet each. It is much larger than any of the others and the placing of the caisson required much care.

Report of the Board of Engineers on the North River Bridge, Senate Document No. 12, 53rd Congress, 3rd Session.

Board of Engineers appointed by the President June 7, 1894:—Major Chas. W. Raymond, Prof. Wm. H. Burr, G. Bouscaren, George S. Morison and Theodore Cooper, p. 5.

P. 40. In proportioning these piers your Board have found it necessary to adopt limits of stress. They have based their estimates on the supposition that the pressure between the metallic bed-plate and the top of the masonry should not exceed 20 tons to the square foot, and that the pressure within the masonry and on the foundation should nowhere exceed 10 tons to the square foot; they consider, however, in determining these pressures that the weight of the material displaced should be deducted. The weight of masonry per cubic foot was taken at 150 lbs. in air, at 87 lbs. in water, at 50 lbs. in mud, and at 30 lbs. in sand. While these pressures have been exceeded in some structures they are higher than usual practice and call for masonry of good quality and more than ordinary cost.

Your Board have assumed that the masonry would finish 50 feet above water and have estimated the cost of these piers, including excavation and sinking at 100 lbs. per cubic foot above a plane 125 feet below water, and have added 8 mills to this price for each additional foot of depth.

Statement of Mr. Charles Macdonald of Union Bridge Company, July 20, 1894, p. 56:—

Cantilever bridge 2,300 feet span. It is proposed to construct this river pier on a sand foundation at a depth 200 feet below water. The diagram submitted herewith indicates the general dimensions and pressures for each of the four cylinders composing this foundation.

It will be observed . . . that the total pressure upon the top of the granite capping is 8·84 tons per square foot, and that the abnormal pressure on the base, when the concrete filling comes into contact with the sand (at a depth of 200 feet), is 7·16 tons per square foot.

The nearest precedent believed to be in existence for a deep foundation of this character is the pier foundation for the Hawkesbury Bridge in New South Wales. The abnormal pressure per square foot in this latter case is 5·7 tons, with a depth of only 8 feet in the sand, and at a total depth of 162 feet below high water.

As it is well known that the resisting force increases with the depth, it is

believed that the assumption herein taken is justifiable, but in order to make sure it is proposed to sink a trial cylinder, 20 feet diameter, in the centre of the square between the four cylinders composing the river pier. From the experimental data thus obtained as to the exact amount of skin friction and resistance to settlement, more accurate proportions can be given to the main cylinders, particularly with reference to the relation of weight required to cause settlement during dredging.

It will be observed that the effect of skin friction has not been considered in calculating the supporting value of the foundations. This will be wholly on the side of safety therefore, and will unquestionably reduce the abnormal pressure on the sand at the foot of the cylinder. These cylinders will be filled up with concrete made of the best Portland cement, lowered through the water in the most approved manner and finished off at about the level of the bottom of the river. The outer skin of the cylinder will be carried up above high water temporarily, to facilitate the construction of the masonry from the river bottom upward.

It is proper to state that what is called "granite masonry" consists of a 4-foot ring of cut granite coping, the interior to be made up of large irregular masses of stone, set in concrete, exactly as was done in the case of the piers for the "Forth Bridge."

Notes as to Pressures on Masonry and Foundations, p. 57.

References :—

Collingwood, "Masonry East River Bridge," Trans. Am. Soc. C.E., vol. vi, pp. 8 and 9.

Cresy, "Encyclopedia of Engineering," 1847, pp. 705 and 706. Quoted from Rondelet, "Traité d'Architecture."

Julius Newman, "Cylinder Bridge Piers," approximate safe loads per square inch.

Gaudard, "Foundations."

Leslie, "Trans. Inst. C.E.," Jan. 24, 1888.

Engineering News, March 14, 1885.

CRESY, "Encyclopedia of Engineering," see p. 4 of notes (i.e. above). Dated 1856, called a new Edition. Preface date 1847.

P. 239. Quay at Rouen, built by De Cessart. The great road from Paris to Havre and Dieppe, France, passing along the ancient quay of Rouen; it was found inconveniently narrow, and in 1779 a new quay was completed, 120 feet in advance of the original wall.

The total length of the new quay was 110 toises; this was divided into seven equal distances by caissons 66 feet in length, 16 feet wide and 14 feet high, their base containing 1056 square inches. Ninety-two piles were driven, 3 feet 6 inches apart in the thickness of the wall, and 3 feet in the length of the caisson, each pile the weight of 18,633 lbs. for the wall alone and adding a half more for the weight of the merchandise placed on it, would then have 27,000 lbs.

All the piles were from 12 inches to 15 inches thick, driven with a ram weighing 1,200 lbs., falling 20 feet, each pile receiving a percussion equal to 300,000 lbs. The heads of the piles were cut off 6 feet below low water at 12 feet behind the wall; piles were driven at every 6 feet to attach land ties, which supported the masonry; sand and gravel were then thrown from the inside of the wall to form a slope on the river side of 60°, which extended 60 feet into the river, so that vessels of 400 tons could approach the quay at low water.

P. 705. As an example of the smallest surface of the points of support of Gothic architecture, we may cite two columns in the church of Toussaints at Angers;

their diameter is only 12 inches and their height 25 feet; they support pointed arches, the mouldings of which are in freestone and the weight carried by each is 35 tons.

(Report of Board of Engineers, p. 57, says 25 tons instead of 35 tons.)

P. 706. We may add from Mons. Rondelet an indication of the pressure exercised on a surface of 9 square inches in the edifices regarded as the noblest. Expressed in lbs. per square inch we have—(1) 113 lbs., (2) 132, (3) 102, (4) 204, (5) 137, (6) 204 practically, (7) 307 lbs.

1. Piers of the Dome of St. Peter's at Rome	1,022
2. " " " St. Paul's, London	1,190
3. " " " The Invalides	922
4. " " " Ste. Geneviève	1,840
5. Columns of St. Paul's without the walls	1,235
6. Piers of the Tower of the Church of St. Méry	1,837½
7. Columns of the Church of Toussaints d'Angers	2,767½

All these figures are lbs. avoirdupois.

In the pier of the Chapel House at Elgin, the stone supports a weight of 51½ tons on each 9 square inches (825 tons per square foot). The stone, which is a red grit, has resisted this pressure for several centuries.

From Board of Engineers' Report:—

P. 58.—(JOHN NEWMAN, "Cylinder Bridge Piers" approximate safe loads per square foot.)

Firm sand in estuaries and bays, 5 to 5·6 tons. Dutch engineers consider safe loads on firm clean sand 6 to 6·16 tons. Very firm compact sand foundations at considerable depth, not less than 20 feet, and sandy gravel 6·7 to 7·84 tons. Firm shale and clean gravel 6·7 to 8·96 tons. Compact gravel 7·84 to 10·08 tons.

Clean sand, homogeneous Thames gravel has been weighted with 280 cwt. per square foot at 3 to 5 feet below the surface and showed no signs of failure—15·68 tons.

GAUDARD, "Foundations."

Stiff clay, marl, sand or gravel, 55 to 110 cwt. (3·08 to 6·16 tons). Gorai Bridge (close sand), Lock Kew (gravel), Bordeaux (gravel), 165 to 183 cwt. Nantes (sand) 152 cwt., some settlement. Szegedin (clay and sand), 133 cwt. (7·4 tons), reinforced by driving piles in interior of cylinder and sheathing outside. Charing Cross, 159 cwt., including adhesion (8·9 tons). Cannon Street, 117 cwt., including adhesion (6·5 tons). Roque Favor aqueduct, 258 cwt., rocky ground (14·4 tons).

Appendix D.—Design by G. Lindenthal, Chief Engineer (p. 74), Suspension Bridge, span of 3,100 feet centre to centre of towers. The river between pierhead lines is here 2,740 feet wide. The New Jersey Tower is located close to the New Jersey pierhead line. The New York Tower is located 150 feet back of the New York pierhead line to shorten the New York end span, and to avoid deeper foundations.

Towers.—The tower bases are hollow, of masonry, reaching 40 feet below high water and extending 30 feet above high water.

The construction of the Tower foundation on the New Jersey side, 90 feet down to rock, is by the usual pneumatic method, using a wooden caisson 175 feet by 335 feet, with two hollow spaces each 90 feet square; where there is no pressure, from the steel columns the air-chamber, after reaching firm bearing, to be filled with packed sand and gravel and with concrete where necessary.

The wooden caisson is of cellular construction; one-third of the section consists of gravel and sand filling. Bearing area 40,000 square feet.

Pressure on foundation 80,000 tons from tower base, after deducting displacement, 76,500 tons superstructure and steel tower complete for fourteen tracks. 61,000 tons extreme live load from fourteen tracks. Total 217,500 tons.

	Tons.
Pressure per square foot	5,425
From wind-pressure of 3,000 tons on tower on leeside	0·200
Maximum per square foot	5,625
From dead load above, per square foot	3·92

Maximum pressure on timber 80 lbs. per square inch.

The New York Tower foundation, 190 feet down to rock, is of a different construction. An open, braced caisson, or cofferdam, with lower edge conforming to contour of rock, as obtained by borings all around, 350 feet by 180 feet inside, of wood and iron 10 feet thick, filled with gravel, is first sunk and the inside dredged out down to rock, which is levelled off with concrete in bags and finely broken stone below water. A hollow-spaced wooden crib 345 feet by 175 feet and 150 feet deep is built up floating inside the caisson. Two large hollow spaces, 75 feet square, enlarging toward the top to 90 feet square, are spread out in the centre of each half tower, where there is no pressure from the steel columns. Masonry below water is also built with hollow spaces. The whole mass of foundation is calculated to float during construction, so that all masonry can be done above water till the whole settles down evenly upon the levelled foundation. All hollow spaces in the wooden crib are then filled with gravel and sand, and in the masonry with concrete.

Maximum pressure on rock foundation, New York Tower, 130,000 tons; tower base, after deducting displacement, 75,000 tons; superstructure and steel tower 61,000 tons extreme live load. Total pressure 267,500 tons on 50,000 square feet or 5·35 tons per square foot; from extreme wind-pressure 0·26, maximum pressure 5·61 tons per square foot; from dead load alone, 4·13 tons per square foot. Maximum pressure on timber 80 lbs. per square inch.

Appendix C 2.—Statement of Mr. W. Hildenbrand to the Board of Engineers (p. 70). A modified plan and estimate for a suspension bridge, p. 73. The towers were calculated for the combined maximum load, and wind strain at 12 tons per square inch.

	Tons.
Weight on top of tower (requiring 7,080 square inches)	84,940
,, of tower	15,110
,, of land truss resting on tower	1,400
Wind strains	1,350

Pressures at base of tower, 102,800 tons requiring 8,566 square inches. Average section 7,823 square inches.

Weight per linear foot, 52,150 square inches.

Total weight of two towers, 579 feet high, 30,200 tons.

The pressure on the rock foundation of the east tower will be 247,380 tons, and the buoyancy of the pier cylinder 90,250; hence the foundation must contain 15,713 square feet in order to resist a pressure of 157,130 tons.

This area can be procured by sinking eight cylinders of 50 feet diameter, filled with concrete, one for each tower column. The total mass of foundation-work

will amount to 2,081,300 cubic feet. The pressure on the West Tower is less, owing to the light inclination of the back cable; hence concrete-filled cylinders of 47 feet diameter will answer the conditions of the foundation.

The total pressure is 137,710 tons, requiring a foundation mass of 1,958,600 cubic feet.

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"Buildings, Foundation of." S. Auglin. Discusses what constitues a good foundation and the working loads different foundations are capable of sustaining. 4,500 w. Stone, Sept. 1900.

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"The Foundations of New York Cathedral." Brief account of the difficulties encountered and overcome in laying the foundations for the new cathedral at Morningside Park, N.Y. 900 w. "Canadian Architect," October, 1896.

"Chicago Post Office Foundations." Review of Paper by Wm. Sooy Smith on the remarkable long-pile foundations of a 300-foot by 360-foot building. Each pile carries 30 tons. 3,600 w. "Engineering Record," November 26th, 1898.

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"Concrete Steel Foundations of the Franklin Building." Illustrated description of unusual concrete and steel grillages. 500 w. "Eng. Record," April 28th 1898.

"Deep and Difficult Bridge Foundations." George E. Thomas. Extracts from

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"Foundations of the East River Bridge, New York." Ill. Particulars showing the progress thus far made. 2,200 w. "Sci. Amer.," August 7th, 1897.

"Eccentric Loads on Foundations." F. L. Douglas. 700 w. "Eng. News," May 20th, 1897.

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"The Distribution of Pressure in Stepped Foundations" (Ueber die Druckvertheilung in Absatzweise Verbreiterten Mauerwerks-fundamenten). Manner in which the pressure of a pier is transmitted to the extended portion of the foundation. 2,500 w. As above, 26th March, 1897.

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"Exchange Court Building, New York City." Describes and illustrates the pile and cantilever foundations of a twelve-storey office building. 500 w. "Eng. Record," 11th June, 1898.

"Foundation Construction for Tall Buildings." Charles Sooysmith. Ill. Principally caisson method. 3,800 w. "Eng. Mag.," April, 1897.

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"Failure of Building Foundation Piers." Partial failure of a new eight-storey, 120 by 110-foot commercial building in Brooklyn. A foundation that failed and the lesson it taught. B. E. Chollar. Read before the Western Gas Assoc. Explains the cause of unequal settlement which damaged a gas-holder tank in St. Louis. 2,200 w. "Am. Gas Light Journal," 3rd July, 1899.

"The Fall of the Gumpendorf Slaughter-House in Vienna " (Der Einsturz im Gumpendorfer Schlachthaus in Wien). Illustrated description of the wrecking of a large building by the slipping of the foundation of a retaining wall. 1,000 w. Zeitschrift des Oesterreichischen, etc., 26th Feb., 1897.

"Deep Bridge Foundations, Atchafalaya River." C. H. Chamberlain. Facts regarding this work in an alluvial section where unusual depth of the piers was made necessary by the instability of the soil when acted upon by river current. Diagrams. 3,200 w. Journal Assn. of Eng. Societies, September, 1898.

"Experiences in an Engineer's Practice." Walter V. Rice. Providing against settling in insecure foundations at the Petrie Bridge, Cleveland, Ohio. 2,000 w. Journal as above, March, 1896.

"Diamond Drill Borings for the New East River Bridge Pier Foundation." 700 w. "Eng. News," 24th Sept., 1896.

"Foundation for the Brooklyn Tower of the New East River Bridge Pier Foundation." 2,800 w. "Eng. News," 27th May, 1897.

"New East River Bridge Foundations." 2,500 w. "Eng. Record," 6th Nov., 1897.

"New East River Bridge Foundations." 2,000 w. "Eng. Record," 5th Feb., 1898.

"Foundation of the New Croton Dam." 7,500 w. Pro. Am. Soc. C.E., April, 1900.

"Foundation of the New Croton Dam." 18,500 w. Pro. Am. Soc. C.E., Jan., 1900.

"The New Croton Dam." Method adopted in putting in the foundation. 1,100 w. "Eng. Record," 7th Jan., 1899.

A Few Facts about the Caisson of the East River Bridge. By K. COLLINGWOOD, Trans. Am. Soc. C.E., vol. 1, p. 353 (1872).

Side Friction.—There was but one time during the whole descent that we had any opportunity to judge even approximately how much it might be. At this time the caisson had been at rest for several days on account of repairs to cars and machinery. Every effort had been made to start it down; the blocking eased so as to have but slight pressure upon it, the shoe carefully examined, etc., and yet no movement took place, until the fall of the tide or a slight variation in air-pressure started it. The average pressure for the day was 17 lbs. per square inch, giving a total lifting force of 20,400 tons. The bearing surface (posts and frames) was about 125 square feet, which, at 5 tons per square foot, gives 625 tons. If to this be added as much more for the pressure on the edge of the caisson, the total upholding force was 21,650 tons. The weight at the time was estimated at 27,500 tons. If these figures are correct the weight upheld by friction was 5,850 tons. The exterior surface of the caisson contained about 13,000 square feet. Hence the friction per square foot was 900 lbs. It is necessary to state, however, that these figures are quite problematical, as there may have been, at points unnoticed, more pressure on the shoe or block than has been allowed for. Even if these figures are correct, it would never be safe to rely upon the side friction as a means of support, except in homogeneous material, on account of its great irregularity.

Concerning Foundations for Heavy Buildings in New York City. By CHARLES SOOYSMITH. Trans. Am. Soc. C.E. 35–459.

It may be well to state here that in the case of the Manhattan Life Insurance building, the weight of which is borne on fifteen caissons proportioned to carry a pressure at their bases of 10·8 tons per square foot, some of them rest on the solid rock and some on the compact stratum mentioned, and that not the slightest settlement can be discovered. As to its safe bearing capacity, there can be no question that it is well in excess of what under the present building law may be put upon it by means of a concrete base, 150 lbs. per square inch or 10·8 tons per square foot.

P. 47. George B. Post, Esq.

Near the corner of Broadway and Ann Street, where the speaker was erecting a high building, there are many lofty structures. The New York Post Office, a heavy granite building, is directly opposite the "Mail" and "Express," and the Vanderbilt buildings, Temple Court, The Potter, "Times," "World" and "Tribune" buildings all stand on sand to which they transfer a pressure of over 4 tons to the square foot without causing any serious settlement. In the case of the World building the blank wall on the east side imposes a pressure of 4·7 tons on the soil and the foundation is loaded eccentrically.

Fall of the Western Arched Approach to South Street Bridge, Philadelphia, Pa. By D. McN. STAUFFER. Trans. Am. Soc. C.E., Vol. VII, 264 (1878).

The ground underlying this approach is an alluvial deposit, treacherous and unstable in character. It is flooded to a depth of about 2 feet at high tide.

At the west bank of the Schuylkill this deposit is about 60 feet deep to the rock beneath (a micaceous gneiss). The rock rises rapidly towards the west and comes very nearly to the surface at a point just west of the arched approach. Overlying the rocks is a stratum of very hard gravel, about 20 feet thick at the river, and above this gravel is mud, which becomes more dense as the depth increases, passing into a tough clay before it reaches the gravel. Running through this mud deposit at different depths are strata of very hard gravel from 6 to 18 inches thick.

After piles of pine and oak had been shattered by the effort to drive them through the thinner strata of hard gravel spoken of, Nova Scotia spruce pine was adopted. This timber is very tough and difficult to split. The piles were from 12 to 18 inches in diameter at the butt, and from 46 feet to 24 feet long when "cut off." The longest piles were, of course, used nearest the river, and were the first driven. The shortest piles were under pier No. 2 (which failed) and the last driven.

A steam pile-driver was used, with a 2,000-lbs. hammer and a 32-foot drop. The only test as to the stability of the pile when driven was to persistently hammer away until after the repeated trials it was found impossible to force the pile any further.

There were eighty-four piles under each arch pier, four rows of twenty-one piles each, spaced 2 feet apart from centres. These piles were cut off 2 feet below low-water mark, and the material excavated from between them to a depth of 2½ feet below the heads, and this space filled with concrete, etc.

The greatest load that can be assumed as being carried by the piles under any

one of the arch piers is 2,000 tons; and this load distributed over eighty-four piles would give about 24 tons per pile, little more than half their safe load, assuming the piles to be driven to a solid foundation.

Pier No. 2 it is reported first began to show signs of failing in the early part of 1897. (The pile-driving was finished in 1870.) In about one year—to the date first given—the north end of the pier had settled 20 inches below its normal position. The south end practically stood firm.

About 7 A.M. on Sunday, February 10th, 1878, the crippled arches gave way at the haunches and fell.

It is a matter of record that the piles under pier No. 2 were from 28 to 30 feet long, as hoisted into the "driver," and that the "average length" of the eighty-four piles as cut off was 25 feet.

Remarks, etc., on above by J. G. Barnard, Trans. 9–319.

The soundings which were made subsequently to the fall showed rock at from 36 feet to 40 feet below surface of the marsh over which the approach was carried. A depression in the rock surface, extending from under the middle of the northern half of the pier and beyond it northerly, was filled to a depth of about 3 feet with "soft mud" mixed with gravel; next above was a stratum of very hard gravel and tough clay 7 feet thick over the mud socket, but only 3 feet thick towards the south end of the pier, where it was in immediate contact with the underlying rock. Thence, upwards, was mud, quite soft at surface "but passing into a tough clay some time before it reached the gravel."

The fall of the structure finally ensued—not, it would seem, as a direct effect of vertical settlement merely, but from the angular displacement of the piles, reaching a limit when they could no longer offer, even temporarily, resistance to the enormous lateral thrust exerted on them. The pier finally sank down (moving southward) by the overthrow of the piles, as the location and conditions of the parts after the fall clearly indicate.

The case presents an almost isolated instance of the fall of the pile-supported structure, accompanied with the overturning of the piles. In this case it deserves the attention of engineers.

Pneumatic Foundations. By General WILLIAM SOOY SMITH. Trans. Am. Soc. C.E., II, p. 411 (1873).

(The first instance in the United States of the sinking of a pneumatic caisson.)

From the Wangoshance Lighthouse I went in 1869 to Omaha to sink the pneumatic pile piers for a bridge over the Missouri River at that point. These were the first pneumatic piles ever sunk in this river, and indeed the first west of the Alleghany Mountains. They were to be put down to a greater depth than, up to that time, had ever been reached by this process anywhere, namely, 82 feet below the water-surface, mostly through finely comminuted silt, interstratified with thin deposits of coarse sharp sand, and layers of tough blue clay, the latter not exceeding 2 feet in thickness. Next to the bedrock there was a stratum of gravel consisting of well-rounded pebbles from 1½ to 2 feet in thickness. These materials presented the most difficult features met in sinking pneumatic piles.

Red Rock Cantilever Bridge. By S. M. Rowe. Trans. Am. Soc. C.E., Vol. 25, 663 (1891).

P. 688. The timber used for the caisson and crib consisted mainly of what was termed "Oregon Pine" (yellow fir), a most excellent timber, strong and firm, though somewhat coarse-grained, and weighed 35 to 40 lbs. per cubic foot when well dried. The amount used in the different parts was about as follows :—

(P. 674. Caisson, 28 by $57\frac{1}{2}$ feet.)	Cubic Feet.	M. B. M.
Working-chamber, including 3 inches inside casing .	6,880	82,560
Roof, 8 feet thick	12,992	155,904
Crib, including 3-inch casing outside	20,071·5	240,855
Making total amount of timber (neat)	39,943·5	479,319

Iron bolts, stay and drift and steel spikes, 29 tons = 58,000 lbs.

	Cubic Yards.
Concrete (beton) put into crib, 47·7 cubic yards per running foot	2,290
" " " " working-chamber	580
Total	2,870

Weight at 4·050 lbs. per cubic yard saturated.

Timber at 35 lbs. will absorb water to nearly 80 per cent., so that the timber 35 lbs. when dry, and absorbing 80 per cent. ($80 \times 35 = 28$) will weigh $35 + 28 = 63$ lbs. per cubic foot, or have about the same weight as water. Then taking the caisson at the time it stopped, we have 10,800 square foot of surface on which the pressure of the material on the outside tended to produce friction, the excess of the weight of the concrete in the curb over the displacement of the water, which, being 101·203 cubic feet at 62·5 lbs. per cubic foot, weighed 3,162 tons. At this time there were also 125 cubic yards of masonry on the crib equal to about 800 tons. Therefore we have—

	Net Tons.
2,290 cubic yards of beton in crib	4,637·25
125 " " masonry	804·00
Total	5,411·25
Resisting this is the uplift of the water	3,162·00
Leaving the downward pressure	2,279·25

equal to about 420 lbs. per square foot when the air-pressure is entirely off. When the air was on at 27 lbs. per cubic inch this tendency downward was entirely overcome.

The main seats for the four pedestals carrying the whole weight of the bridge are 9 feet square and $2\frac{1}{2}$ feet thick, made up, however, of four sections each.

	Lbs.
The anchor arm, including floor, weighs . . .	593,000
The cantilever arm, including floor, weighs. . .	613,000
The two bridge seats, pedestals and piers . . .	79,850
The suspended span, one half including at anchor .	656,700
Live load on all at 3,000 lbs. per cubic foot . .	1,980,000
Making a total weight in pounds	3,943,550

Then, as there are two seats on each pier, 3,943,500 + 2 = 1,971,775 lbs. on each seat. Each seat 7 × 7 feet equals 7,056 cubic inches and 1,476,775 lbs. divided by 7,056 = 279·4 lbs. per cubic inch, or a little over one-sixtieth of the ultimate crushing load (12,000 lbs. per square inch as tested) of this sandstone.

Completion of bridge on June 25th, 1890, by the Phœnix Bridge Co. Prof. Burr and Prof. Waddell were the designers.

Record of sinking Red Rock caisson, given in Appendix "A," p. 712. Total height of caisson, January 29th, 1890 = 63·81 feet.

Computation of Pressure on Piers. By H. H. Quinley.

	Lbs.
Anchor arm, including iron floor	620,700
Cantilever arm, including iron floor	659,600
Anchorage reaction (excess of cantilever over anchorage)	30,000
Pedestal posts and bracing over pier	89,400
Suspended span, including anchorage reaction . . .	702,000
Track at 450 lbs. per foot	297,000
Live load at 3,000 lbs. per foot	1,980,000
Total on pier	4,378,700
Total on one pedestal 7 feet 3 inches square	2,189,350

7 feet 3 inches square = 7,569 square inches, and 2,189,350 divided by 7,569 = 289 lbs. per square inch.

(Material: (1) sand, gravel and boulders, (2) cemented gravel and boulders, (3) compact sand, gravel and boulders, (4) No. 2 again, (5) rock.)

(From Plate cxxi, p. 696.) Outside dimension of pier—East Main Pier—60 feet by 30 feet.

Estimated August, 1889.

Masonry	853·05 cubic yards.
Beton in chamber	384·0 „ „
Beton in crib	1,875·4 „ „
Timber in caisson	32,943 „ feet
„ „	395,320 feet B.M.

Final Result, June 1890.

Masonry.	633·27 cubic yards.

Chimney for the Narragansett Electric Lighting Co., Providence, R.I. By John T. Henthorn, M. Am. Soc. C.E. Vol. XXV (1891), p. 1.

This foundation consists of piling and concrete, and to arrange for it a space of 44 feet square was first excavated 5 feet 6 inches below zero line, or high water, and the sides protected by driving 3 feet spruce sheet-piling 16 feet long. Over this excavation the pile-driver, having a ram of 2,200 lbs., was rolled. Spruce piles 50 feet long and spaced 30 inches centre to centre, were driven as far as possible without breaking. There were twenty-three of these chimney piles cut off uniformly at 5 feet below the high-water line, the earth around their heads thus being 6 inches below their tops. The intervening space between the sheet-piling was filled in with concrete. This mass was carried up to the 1-foot 3-inch level, and consequently formed a foundation 6 feet 9 inches thick, with the head of each

pile projecting 6 inches therein. This was then covered with earth and allowed to season during the winter.

On May 31st, 1889, work was resumed by laying the first brick of the chimney. This was carried up in the form of a square of 36 feet, to a height of 3 feet 2 inches, and from that level the base of the chimney proper, which was 28 feet 6 inches square, was started.

Extreme height above high water, 260 feet 9 inches.

P. 7. The amount of material used in the course of construction above the concrete foundation is as follows :—

Brick	1,332,921
Lime	695 casks
F. O. N. cement	1,025 ,,
Portland	17 ,,
Soapstone colouring	99 ,,
Sand	3,858 ,,
Cast-iron cap	22,,000 lbs.
Cast and wrought iron	7,215 lbs.
Copper bolts	250 lbs.
Lightning rod and brass castings	326 ,,

Vol. VII (1878), p. 331.

East River Bridge. By W. A. ROEBLING.—Consists of a central suspended span of 1595 feet 6 inches length between centres of towers, and two side spans, also suspended, each 930 feet long.

The approaches increase the total length to about 1⅛ mile. The ends of the cables are anchored in two masses. These masses are each about 119 feet × 132 feet in plan at the base, and about 89 feet high, and are founded on a grillage of timber from 4 feet to 7 feet thick which rests directly on sand, the timber below the level of water in the soil and consequently not subject to decay.

The piers at either side of the river rise to a height of 271½ feet above mean high tide. At high water surface the extreme measurements were in plan 57 feet × 141 feet in Brooklyn and 59 feet × 141 feet in New York.

In Brooklyn the foundation rests mostly on boulder clay, but a sufficiently uniform foundation was not found until a depth of 44·5 feet below tide was reached. To obtain this depth a timber caisson was sunk, having exterior dimensions of 102 feet × 168 feet and a height of 24·5 feet. Both caissons when launched were 15 feet high, and additional timber to the heights named was put in after launching. The air-chamber, which was afterwards filled with concrete, had a height of 9 feet. The final pressure at bottom of foundation will be about 5½ tons per square foot. The pressure on the top of the timber is 9½ tons per square inch. The total quantity of masonry, including concrete, 43,900 cubic yards.

The New York Pier rests on compact sand and gravel, immediately overlying the bedrock. The caisson was 102 feet × 172 feet × 31½ feet, also of timber. The edge of the caisson is 78 feet below tide. The tower contains about 55,000 yards of masonry and concrete. The pressure at the base is about 6⅔ tons, and on top of the timber about 10½ tons per square foot.

Work on the Brooklyn caisson was begun November 1st, 1869. The caisson was launched in March, 1870, and put in place, and the first stone set June 15th, 1870. It was sunk to full depth and filled in by March, 1871. Stonework of the pier completed December 1st, 1874.

The New York caisson was begun September 6th, 1870, launched May 8th, 1871, put in place and sunk to depth May 17th, 1872, filled with concrete July 22nd, 1872. First stone set October 31st, 1871, and pier finished July 15th, 1876.

Last wire of cable laid October 5th, 1878.

East Pier of the St. Louis Bridge. By JAMES B. EADS, Chief Engineer. P. 332.

All the great piers of this bridge, four in number, stand upon the bedrock of the river. Two of them are nearly 200 feet high. The base of the west abutment was laid within a cofferdam; the other three were sunk through the water and sand by the method of compressed air. The method employed was in many respects entirely new, and in nearly all important respects the work was on a scale far surpassing all previous experience. The most difficult, on account of the depth and strength of the river and the distance to the rock, was the East Pier, and it was undertaken first. The iron caisson enclosing the air-chamber was first towed into position between the large guide piles. Its length was 82 feet, its width 60 feet, and the depth of the chamber was 9 feet.

The East Pier was sunk from the surface of the river to the bedrock in 134 days.

The caisson of the east abutment was built of wood, with only a single thickness of iron plate to make it air-tight. The wood consisted of squared oak timbers, bonded and strongly bolted. This caisson was filled with sand, except under the walls, which rest on concrete. The base of the east abutment is 83 feet × 70 feet 6 inches; the top is 64 feet 3 inches × 47 feet 6 inches. The height of the masonry is 192 feet 9 inches. It contains 22,453 cubic yards of masonry, and its weight, with half the spar it supports, is about 46,500 tons. Its base is 134 feet 6 inches below high water.

The caisson of the East Pier cost $111,000·00, and that of the east abutment $139,700·00. The masonry and sinking of the East Pier cost about $469,000·00; that of the east abutment $451,000·00.

The chambers of the channel piers were filled with concrete.

P. 5. *Detail Drawing of the Foundation Work of Pier No. 5 of the St. Charles Bridge over the Missouri River.* By C. SHALER SMITH, C.E. Vol. VII., p. 335.

This bridge crosses the Missouri River 17 miles west of St. Louis and 20 miles above the junction of the Mississippi and Missouri Rivers. Being within the range of the Mississippi backwater, the variation of the water-level at this point is over 40 feet, and the flood speed of the current very great, exceeding in fact 9½ miles per hour twice during the period occupied in the construction of the bridge. In 1869, after the piles had been driven for the breakwater for pier No. 5, a heavy freshet occurred, which carried away the works at this point, and in doing so scoured out a large hole at the pier site, which hole was soon filled with the travelling boulders which move along the bed of the river at such times. After the subsidence of the flood, a careful examination showed the proposed site was occupied by an inverted pyramid of boulders and drift wood, 200 feet long by 70 feet wide at the base and about 22 feet thick at the deepest part. The caisson, which was of iron, double wall and cellular, had already been arranged for sinking

with the water-jet and Ead's sand-pump, and was altered to suit the new conditions of the case.

The most economical method of working was found to be as follows :—

While excavating, the air was kept at a pressure of from 10 to 15 feet (or 4½ to 6½ lbs. per inch) greater than required at the depth at which the men were working. This dried the bed for 2 feet below the cutting edge. The boulders were excavated until water was reached and under the bearing beams were replaced by sand well tamped. The sand layer was then pumped out and the pressure in the air-chamber lowered until the pier sank down to the top of the boulders. A marked feature of the foundation was the great friction of the materials after the boulders had commenced caving and packing. In the last 20 feet the pier never moved until a skin friction of 466 lbs. of immersed surface had been overcome.

No. 6. *Detail Drawing of the Foundation Work of the Poughkeepsie Bridge.* By P. P. Dickinson, Chief Engineer. Vol. vii, p. 336.

This work is located at Poughkeepsie, 75 miles north of New York City. The width of the river at the bridge site is 2,430 feet and the depth of water from 50 to 60 feet, with a tidal motion of 3 miles per hour. The bed is composed of 20 feet of sediment and mud, 10 to 40 feet of compact blue clay, 6 to 10 feet of sand, and 10 to 15 feet of coarse gravel, with boulder overlying the rock, which is at a depth of from 119 to 145 feet. There are to be five spans of 525 feet each, with a depth of truss of 65 feet. The substructure consists of four river and two shore piers, with two abutments, to be built of granite masonry to a height of 135 feet above high tide, having a base of 72 by 32 feet at 20 feet below high tide, and 40 by 12 feet at top, giving a pressure at the base of about 5·0 tons to the square foot. The shore piers and abutments have their foundation on rock, on the river bank.

The four river piers are founded on caissons, filled with concrete and resting on the bed gravel, the East Pier being 122 feet and the West Pier 97 feet below low tide. The caissons are 60 feet wide by 100 feet long, composed of yellow pine and white hemlock timber, 12 inches square. The lower ends of the end, side and central portions of the caissons are built up solid with timber thoroughly bolted together for a height of 18 feet, running from a cutting edge, shod with iron to a thickness of 10 feet on the sides and ends and 15 feet in the centre wedge-shaped portion. Transverse walls of timber, commencing 4 feet above the cutting edges, bind the caissons together, dividing the central portion into twelve open compartments at a height of 16 feet; the transverse and the exterior longitudinal walls are 3 feet thick, and the four interior longitudinal walls 2 feet thick. By this plan of building the caisson is divided into forty compartments. The outside and centre line of compartments are twenty-eight in number, placed over the cutting edges. These compartments receive the concrete required for additional weight in the sinking. The remaining twelve compartments, each 12 feet square, extend to the bed of the river and through them the material is removed by the aid of the Clam Shell Dredge, the sinking of the caisson being controlled by excavating from either of the twelve compartments until finally resting in position.

A notable feature of the work is the great depth from which material has to be dredged and the ease with which the caisson is held in position. The extreme depth of dredging required is 130 feet, which is being done without difficulty.

The caissons contain each an average of 2,500,000 feet B.M. of timber, and 350 tons of wrought iron, and will contain forty columns of concrete, twelve of

which, 12 feet square, extend from the bed gravel to within 20 feet of low tide, the remaining twenty-eight resting upon the cutting edges, which are solidly embedded in the gravel and extend to the same height. The concrete is composed of Portland and Rosendale cement, mixed with sand, gravel and broken stone in the proportion of 5 to 1.

The use of Compressed Air in Tubular Foundations and its Application at South Street Bridge, Philadelphia, Pa. By D. McN. Stauffer, C.E. Trans. Amer. Soc., vii, p. 287 (1878).

Site of the Bridge.—The Schuylkill River, where the bridge crosses it, is 467 feet wide from bank to bank, at low-tide mark, with an average depth of water at low tide of 7 feet 6 inches. To the west of the river was a low marshy meadow, extending some 500 feet inland, and subject to overflow at high tides.

The rock upon which the cylinders were to be founded was a micaceous gneiss, dipping to the north-west, and as mentioned above was very irregular in surface, and generally soft and shelly on top. Above the rock was a deposit of, first, hard gravel, then sand, and stiff mud, and then mud, the lower strata intermingled with boulders and some drift wood. This deposit varied at the pier sites in depth from 24 feet to 5 feet.

The river was crossed by a bridge of three iron through spans; two of them, each 192 feet long and 36 feet wide between truss centres, were fixed, and between these two was a draw span 200 feet long and 23 feet wide between truss centres. This arrangement required three piers, one of them a pivot pier, in the river, the centre of the pivot being 99 feet distant from the centres of the pier east and west of it, measured on the centre line of the bridge. The footways were outside of the trusses and made the extreme width of the 55 feet bridge on the fixed span and 36 feet on the draw span. The distance from the top of the bridge floor to low-tide mark was 42 feet.

Pier Cylinders.—The piers were made up of thirteen cast-iron cylinders, extending from the rock to the superstructure of the bridge, arranged as follows:— Two iron cylinders, each 8 feet in diameter, located 36 feet apart between centres, in a line at right angles to the centre line of the bridge, formed a support for the river end of each fixed span, and for the end of the draw, when closed. The pivot pier was composed of a central column 6 feet in diameter, and eight columns each 4 feet in diameter, surrounding it, their centres located on the angles of an octagon. The outside diameter of the pivot cluster was 36 feet, the centre of each 4-foot column being 16 feet from the centre of the 6-foot column.

Material of caissons—next given.

Plant used in sinking caissons—help, etc.

(Plenum process.)

Filling the Columns with Masonry.—The columns were all filled to the top with rubble masonry, laid dry and thoroughly grouted, with hydraulic cement grout, every 3 feet in depth.

The first 10 feet of masonry was laid under pressure—that height of stonework being generally found sufficient to seal the column against the entrance of water from below.

Rate of Progress in Sinking the Cylinders.—The material through which the cylinders passed at South Street was a sandy mud at top, gradually becoming more compact and tough, with but little sand, as the depth increased, with a

stratum of hard gravel 3 feet to 4 feet thick just before the rock was reached. Large boulders and driftwood were scattered throughout the two lower strata of the deposit.

P. 305. *Effect of Frost upon the Cylinders.*—The same mistake was made at South Street as at Harlem Bridge and at Omaha, and in fact nearly every place where iron tubular foundations have been used in the States. No precautions were taken to prevent the cylindrical sections from splitting, by the contraction of the metal from cold, upon the mass of grouted masonry within. At South Street four or five of the sections were cracked from this cause the first winter, some of the sections splitting horizontally and others vertically. The worst crack was a vertical one and opened nearly ¼ of an inch, but it has never increased since, and seems to have done no further harm. It has been recommended, as a means of avoiding this trouble, to line the inside of the cylinder with staves of pine or other soft wood a little thicker than the flanges are wide. In this case the wood would crush under the strain and tension upon the metal be relieved.

Load upon the Pier Columns.—The maximum work upon the stonework inside the cylinders is an unusual one for what might be called rubble masonry, and is worth noting.

Taking the pier for the fixed span, and end of draw formed of 8-foot columns, the maximum load upon it may be divided as follows :—

	Lbs.
Half weight of the fixed span	412,225
Quarter weight of the draw	175,542
Half load on fixed span, at 80 lbs. per square inch .	400,000
Quarter load on draw span at 80 lbs. per square inch	70,000
Total load-maximum . . .	1,057,767

The diameter of each of the columns of masonry upon which the load is thrown is 7 feet 9 inches, equivalent to an area of 94·34 square inches in the two columns, and the area divided into the total load would give 11,224 lbs. or, just 5 tons per square inch on the masonry at the top of the column. The longest cylinder is 71 feet 9 inches; taking the masonry at 130 lbs. per cubic foot the weight of this column of masonry, acting upon the lowermost course, would equal 9,327 lbs. or 4·16 tons per square inch, in addition to the weight of the bridge, or in other words there would be 9·16 tons per square inch upon the bottom course of masonry in the cylinder.

With the draw open the load upon the 6-foot column supporting the pivot is theoretically much greater; this pivot was designed to be centre-bearing, but fortunately, perhaps, this design was a faulty one; a considerable but unknown portion of the load is actually thrown upon the outside wheels and then transferred to the 4-foot columns.

But supposing the whole load to be carried by the central column, as was intended, the conditions would be as follows :—

The total weight of the draw span, unloaded, is 702,168 lbs. and the area of the masonry column, 5 feet 9 inches diameter, is 25·9 cubic inches. This would amount to 27,110 lbs. or 12 tons per square foot upon the top of the masonry, 64 feet high, to the above 3·7 tons per square foot, or a total load of 15·7 tons per square foot.

None of the columns have shown the slightest sign of weakness, though the bridge has been heavily loaded. As before remarked, the quality of the cement

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and the stonework was very good, thanks to a conscientious foreman, for it was impossible for the engineer to watch all of the inside work. The stone used was a hard well-bedded limestone.

The Raz-Tina Lighthouse. By REYNOUL. Proc. Inst. C.E., vol. cxxx, p. 341. From "Annales des Ponts et Chaussées," 1897, p. 252.

This lighthouse is situated off the port of Sfax, and is, together with the buildings connected with it, constructed entirely of concrete. The base is on a small rocky mound rising on to a plain of sand only slightly above sea level.

The foundation block is 39 feet 4½ inches square and altogether 13 feet 1½ inches high. The tower has an internal diameter of 5 feet 11 inches, an outside diameter of 10 feet 6 inches at top and 27 feet 10½ inches at bottom, the platform being 137 feet 4 inches above the foundation block, the height over lantern above foundation 151 feet 6 inches, and the centre line of the light being 181 feet 9 inches above high-water level.

The pressure of the foundation on the soil was calculated at 27 lbs. per square inch (about 2 tons net per square foot), and the maximum pressure on the concrete of the tower at 64 lbs. per square inch, allowing for wind pressure 51¼ lbs. per square inch and 131 lbs. per cubic foot for the weight of the concrete. During construction the base settled, quite evenly, to an extent of 5½ inches. The total cost of the lighthouse complete was £3,440, of which £2,560 was for the tower and buildings.

The Blackwall Tunnel. By DAVID HAY and MAURICE FITZMAURICE. Proc. Inst. C.E., vol. cxxx, p. 50, London, Eng.

P. 54. The caissons are 48 feet in internal and 58 feet in external diameter, and are formed of two skins partly of steel and partly of iron.

The shafts on south side of river were sunk by pumping down the water and removing the excavation from the inside in the ordinary way. The weight of caisson and concrete filling was generally sufficient to overcome the skin friction, until about 20 feet from the bottom, when more weight had to be added temporarily.

(London clay and sand and gravel were penetrated.)

The friction on the outside skin of No. 3 for the last 20 feet amounted to about 5·6 cwt. (627 lbs.) per square foot, the cutting edge being entirely free.

On the north side of the river No. 2 shaft was sunk.

From four borings taken round the site it was ascertained that the whole of the ground to be penetrated consisted of ballast.

The total weight of the caisson and sinking load amounted to nearly 4,000 tons, the skin friction being therefore about 4½ cwt. per square foot of outside surface.

The Greenwich Footway Tunnel. By WILLIAM C. COPPERTHWAITE. Proc. Inst. C.E., vol. cl, p. 1, for 1902.

The second tunnel undertaken by the London County Council with a view to improve the means of communication between the districts lying north and south of the river Thames and east of the Tower Bridge.

P. 3. The two shafts are alike in general construction and differ only in depth, that on the Poplar shore measuring 60 feet 4 inches from top to cutting edge and the Greenwich shaft 66 feet 8 inches.

Caissons.—The caissons are 35 feet in internal and 43 feet in external diameter, and are formed with two steel skins, the 4-foot space between which is filled with 6 to 1 Portland cement concrete.

The erection of the caisson was commenced on the 26th September 1899, and by the 13th March, 1900, one-half was erected and the cutting edge was sunk to a depth of 24 feet 6 inches below ground level. The material passed through was mainly river mud and silty clay, but at 22 feet below the surface ballast was found, through which, as already stated, water found its way from the river, the water in the caisson rising and falling with the tide. (Sank from then on by compressed air; before that in the open.) Below the ballast, at about 41 feet below ground-level, was found a bed of close grey sand, at times almost as tough as soft sandstone. It was noticed that the skin friction of the caisson which, when the lower portion was in the ballast, had been generally 4·5 to 4·7 cwt. per square foot (say 575 lbs.), became less as the cutting edge sank deeper into the sand, and the last observation, taken when the caisson was nearly down to the required level, gave a skin friction of just under 3·8 cwt. (426 lbs.) per square foot. The probable explanation is that, owing to the consistency of the sand, most of the air escaping from the pressure chamber passed up close to the outside of the caisson and so made an air lubricant for it.

The total weight of kentledge put into the caisson was only 921 tons, which, with the weight of steel and concrete in the structure, made finally a total weight of 2,560 tons.

P. 74. With regard to the skin friction, the Table, p. 8 (and statement as on preceding page), was perhaps hardly definite enough . . . . Of course he did not mean that the pressure given in the column had been the pressure exactly at the moment when the skin friction had been taken. Supposing the pressure to have been 16 lbs. per square inch and the excavation to have been taken out to the extent shown in Fig. 4, Plate 1, the custom had been to lower the pressure gently and a drop of about 4 lbs. per square inch in the pressure had generally been found enough to make the shaft bury the cutting edge in the sand; it was on these figures that the skin friction had been worked out.

P. 66. Mr. James Brown remarked that, while the Greenwich shaft had been sunk with the same regularity as the Poplar shaft, after it entered the shelly clay it had become very difficult to move, and that difficulty had increased the lower the shaft was sunk. So long as it had been in sand and ballast the skin friction had worked out much the same as at the Poplar shaft, namely 4 cwt. to 4½ cwt. (448 to 504 lbs.) per square foot, but after entering the shelly clay it had risen to 6 cwt. (672 lbs.) per square foot of surface, and when within 1 foot of the bottom, to 8 cwt. (896 lbs.). At the last attempt to sink, with only 2 inches further to go, the load had been increased to 8·17 cwt. (915 lbs.), and then the shaft had declined to move. Therefore he thought possibly the caissons might have been better with a slight taper on the outer skin, or perhaps a mistake had been made

in discontinuing the use of lubricating-pipes. These pipes had been fitted on the Poplar shaft but had not been required there, as the shaft had sunk freely in the soft sand, and they had been used only occasionally, more by way of experiment than anything else. Mr. Copperthwaite said "without appreciable results." He had been very sceptical himself about deriving any benefit from their use, as he had expected that the compressed air escaping round the cutting edge of the shaft would merely force back the water into the ground and leave the skin dry. But he had calculated the skin friction for the first day on which the pipes had been in use, when presumably they had been in perfect working order, and he had obtained a better result on that day than on any other occasion. On the 22nd of May, before fitting the pipes, the skin friction had been 4·26 cwt. per square foot. On the 24th May, when the pipes had been first used, the skin friction had fallen to 3·61 cwt., and on the 30th May, after discontinuing the use of the pipes, it had risen again to 3·94 cwt. The pipes seemed therefore to have had quite a considerable effect, but as the water had been drawn from the river-bed, the pipes had soon become choked and useless, and it had been decided not to fit them on the Greenwich shaft.

Pile-sinking by means of a Hydraulic Jet at Moruya and Carrington Bridges, New South Wales. By Ernest M. de Burgh. Proc. Inst. C.E., vol. cl, p. 340.

The difficulty frequently experienced in sinking screw-piles has in some instances led to the employment of a water-jet to facilitate the work of sinking. In the structures described in the following Paper, the Author adopted cast-iron piles on account of their durability in teredo-infested salt water. The section of the Moruya River, at the point where the South Coast Road crosses it, shows soft granitic rock underlying sand and clay at depths varying between 4 feet and 36 feet. The bridge which carried the traffic up to the year 1889 consisted of 16 timber spans of 50 feet each, supported on hardwood piles. In the new structure 18 spans of 45 feet each and 2 spans of 25 feet each were adopted, as these spans admitted of the use of corbelled iron-bark beams, and it was necessary to construct 19 piers and 2 abutments. The deck is 18 feet in width between the curbs.

General Type of Pier adopted.—Each pier consists of three cast-iron piles 12 inches external and 10 inches internal diameter. Each pile was cast in lengths of 12 feet or less to suit the conditions of sinking, and the connections between the lengths of piles were made by means of outside flanges thick enough to admit of the bolts being housed into them, so that after the bolts had been screwed up with a box spanner they could be flushed over with Portland cement to protect them from the action of the salt water.

It was considered desirable to sink the piles of piers 4 to 14 (33 piles) to a depth of 18 feet to 37 feet, and to fix those of piers 1, 2, 3, 15, 16, 17, 18, 19, and of the abutments (30 piles) in the rock, in holes excavated for the purpose. It is believed that the piles of piers 4 and 9–14 reached the rock and rest upon it.

Type of Shoe.—The maximum load on each pier pile in the finished structure amounts to 24 tons, with a full live load of 84 lbs. per square foot of deck, and on the abutment piles to 9 tons. Whilst the cast iron columns with flanges, when sunk to a depth of 35 feet in sand, would probably sustain such a load safely, it was considered desirable to increase the bearing surface by the addition of a shoe, which would also act against overturning in flood by resisting the lifting of the upstream piles.

It was found that a central orifice of 2 inches diameter and 4 orifices of $\frac{1}{2}$ inch diameter . . . . provided the most effective arrangement in the sand and soft clay, with occasional sand bands which were met with; but, as will be seen later, difficulties were encountered when the depth of clay to be penetrated exceeded 11 feet 6 inches and the clay itself became more compact.

The time occupied in sinking a pile through the sand overlying the clay, an average depth of 14 feet, after it was placed in position, averaged 6 minutes, inclusive of the time occupied in shifting the gear. The actual rate of movement was about 8 feet per minute. No appreciable variation was found in the speed or ease of sinking in sand at various depths until the clay was reached, nor was any great difference noticed at the Carrington Bridge. It is of importance to note however that the fact of the sand being covered by water makes sinking much easier than if the surface is dry. There is no doubt that this limit (of 11 feet 6 inches) to sinking in the clay may be attributed to three causes: (1) the compact nature of the clay met with at that depth; (2) the resistance caused by the strata closing in under the flange joints of the pile; (3) the failure of the shoe to clear well. In the case of the piles (12 in number) of piers 5 to 8 inclusive, the average depth of 9 feet 3 inches which they reached in the clay under a 10-ton load and the action of the pump was considered sufficient on account of their position in the current of the river. These piles were loaded, as described later, to test their bearing power only, but in the case of the piles (18) of piers 9 to 14 inclusive, it was considered desirable to press the sinking as far as possible. With this object in view the pumping was continued until an average depth of 13 feet 8 inches in clay was reached under the 10-ton load, and movement practically ceased. The pump however kept working for 2 to 3 hours longer and additional loading up to 32 tons per pile was applied, which produced an average penetration (additional) of 4 feet 4 inches, making a total depth in the clay of 18 feet on an average for each of the 18 piles. The clay, which was several times exposed and examined during the operations with the air lock, was of a compact nature, with some sand veins near the top, but stiff and tough below. The Author is of the opinion that, for sinking in clay, the shoe used at Moruya, which was designed for sinking in sand, could be modified with advantage, for it would seem that the action of the jet softened and wore away the clay, but that the hole made had a less diameter than the shoe, upon the edge of which the pile remained supported until the added weight broke down the resistance. The substitution of internal screwed joints for the external flange joints used to connect the piles would be found of advantage in deep sinking.

P. 348. *Testing Sustaining Power of Piles.*—It has been stated that the full load (dead and live) on each pier pile was estimated at 24 tons. This load is based on an assumption of a live load of 84 lbs. per square inch of deck area, which is generally adopted as a standard by the Public Works Department of the State of New South Wales, but it is rarely met with in country districts. In the case of the piles of piers 5 to 8 inclusive (12 piles in all), the sinking had stopped in clay, and, as the action of the water jet was to soften and carry away the material below the pile shoe, it was thought desirable to test the sustaining power in each case. A load of 28 tons per pile was accordingly applied and was left in a position for a period of between 24 and 48 hours, or longer in cases where the yield or set reached a maximum. The downward pull when sinking these piles was 10 tons, and it was found that the set under 28 tons was, on an average, 4 inches. The minimum set was $2\frac{3}{4}$ inches; in this case the pumping had been continued for 12 minutes after the pile failed to move under the 10-ton

load; 5½ weeks had elapsed between the date of sinking the pile and that of applying the test load, and the sinking was through 12 feet of sand and 4 feet 3 inches of clay. The maximum set was 10½ inches. In this case the pumping had been continued for 2½ hours after the pile failed to move under the 10-ton load; only 12 hours had elapsed between the time of sinking and the application of the test load, and the sinking included 13 feet in sand and 11 feet in clay. In general, it was found that the set under the test load increased in proportion of clay sunk through, and with the time pumping was continued after the 10-ton load failed to move the pile; while the longer the pile was left before being tested the less it moved.

The Moruya Bridge was opened in 1900, and has since carried a fairly heavy traffic, and has been subject to a flood which rafted timber heavily against it, but up to the present time (November, 1901) no signs of movement or subsidence have been detected.

P. 349. *Carrington Bridge.*—This bridge, like the Moruya Bridge, was erected to replace an old structure. It carries the road traffic from Newcastle to the Dyke. The new bridge has nine spans of 30 feet and one span of 35 feet all of iron-bark beams with a 24-foot roadway, and one footpath 6 feet in width. The piers consist of cast-iron piles similar to those at Moruya, but there are five piles in each pier, the two outer piles being 1 foot in diameter and the three inner piles 9 inches in diameter, all having shoes of the same size and shape as shown. Reference is made to this bridge on account of the sinking being wholly in sand and also on account of the different methods adopted in resting the piles. There are forty-five iron piles in the bridge, those in the abutments, ten in number, being of timber, and assuming a live load of 84 lbs. per square inch of deck, the estimated load on each pile is 15 tons. All of the iron piles were sunk . . . to a very uniform depth of about 20 feet in fine sand containing some layers of shells.

It was decided to weight some of the piles to 18 tons, and the first two piles so tested gave no sign of yielding, although the load was left on for several days; in the case of the two other piles, however, where the depth had been adjusted by the means already referred to, yields of 1½ inch to 2 inches respectively were recorded under the load. As this yield was very small and evidently due to the disturbance of the sand only, and the cost of loading was considerable, experiments were made to ascertain whether the desired result could not be obtained equally well by giving each pile a few light blows with a pile engine which was available. A timber head was placed on the piles to protect them from injury and twenty blows were given with a 25-cwt. monkey, the drop being gradually increased from 3 feet to 5 feet. The whole of the piles were treated in this manner with very uniform results, the general set being about 2½ inches and the maximum set in any case 3½ inches, and it was found, on placing the 18 tons test load on several of the piles after they had been tested with the monkey, that no further subsidence could be obtained. On the other hand, when the monkey test was applied to the piles which had previously been loaded a further set was obtained to an extent sufficient to satisfy the Author that, for the purpose of settling the piles, the monkey test was not only cheaper but also the more reliable of the two.

The Carrington Bridge was completed and opened in August, 1900, and has since carried a heavy and continuous traffic. In no case has the slightest subsidence of the piles occurred.

Notes from the Minutes of Proceedings of the Institution of Civil Engineers.

Subject Index, vols. cxix to cxlvi (1894–1900).

Foundations ; quays ; piers.

Deep-water Quays, Newcastle-upon-Tyne. By ADAM SCOTT, Assoc. M. Inst. C.E. Vol. cxix, p. 291.

The Newcastle quays extend for a length of about 4,620 feet along the north bank of the River Tyne.

P. 293. *The New Quay Wall.*—In 1886 a slip occurred in the quay built in 1840 on the east side of the 60-ton crane. Mr. P. J. Messent and Mr. W. G. Lows, the City Engineer, jointly reported on the rebuilding of the quay, and recommended the construction of a deep-water quay to give a depth of about 20 feet at low water of spring tides, the foundations to be constructed by sinking well-monoliths of concrete.

The wall consists of a substructure of large monolithic blocks, each (with one exception) 30 feet long, 20 feet wide and 37 feet deep, with a well 20 feet by 10 feet, and walls 5 feet thick. The caissons were 37 feet deep, sunk to an average depth of about 32½ feet below low water, and were filled with concrete. They were set about 2 feet apart and the spaces between them were filled with concrete to a certain depth. On these blocks a masonry and concrete superstructure was built. Three monoliths, covering about 94 lineal feet, were sunk for the restoration of the fallen work, which was completed between 1888 and 1890. It was afterwards decided to continue the work westward to the 60-ton crane, a further length of about 188 feet, requiring 5 blocks 30 feet long and one closing block, No. 9, 22 feet long, which was finished between 1890 and 1893.

The crib of the monolith was 6 feet in height, the cutting edge being an iron casting of V-shaped section 2 feet 1 inch deep, with vertical wrought-iron straps attached and timber lining. The bottom was levelled to receive the shoes, and was made up where necessary to 3 or 4 feet above low-water level. Straps were put across the corners on the inside at the top of the curb to prevent the sides bulging out. The curbs being set level were filled with concrete, and on this the sides, 5 feet thick, were built all round. The shutters for connecting were 3 feet deep and were carried on 9-inch by 3-inch standards. After each 3-foot filling sufficient time was allowed for the concrete to set. When the monoliths had been built to a height of 9 or 12 feet above the top of the curb they were stripped and sunk, the interior being taken out by grab dredgers until the top was three or four feet above low-water level. The standards and shutters were then again fixed, the sides built up another 9 or 12 feet, and the caisson sunk, and so on, alternately building and sinking until the full height of 37 feet was attained, with the toe of the curb fairly entered into the hard ballast, which lay at about 31¾ feet below low-water level, leaving the top of the monolith at a permanent level of about 4½ feet above water level. The ballast was generally found to be very hard, the grab often bringing up large crusts resembling concrete.

The sinking of the monoliths was a difficult and anxious process owing to the nature of the strata to be passed through and the danger to the street behind, arising from a bed of quicksand and mud between 19 and 20 feet thick, which caused a large additional amount of excavation rushing in and sometimes filling

up the well to a depth of several feet. Old rails and kentledge blocks were used as weights in sinking the caissons. In the first section, the greatest weight put on a block was 220 tons; for the second section 150 tons of special kentledge blocks were cast, and in addition to these there were at the last 200 tons of rails, making the heaviest load 350 tons.

The spaces between the monoliths were piled at the back and front, and the material within was cleared out by divers and by a small grab made for the purpose, to a depth in the centre of about 27 feet below the top of the block. This process was a tedious and troublesome one owing to material running in from the back or front or from both.

On this substructure was erected the upper wall consisting of sandstone ashlar, facing backed with 1 to 5 cement concrete, to which some rubble was added, and finished with Aberdeen granite coping 4 feet by 1 foot 9 inches. The filling behind the new wall was principally of ashes, but a little of the excavated material was placed at the back of the first length of wall.

Mr. W. G. Laws was the Engineer for the work, Mr. P. J. Messent being the Consulting Engineer. The Author acted as the Resident Engineer.

Apparatus for determining the Safe Load upon Foundations. By R. MAYER. Vol. cxlii, p. 408.

The Author has now designed a hand apparatus which only costs £5. It consists of three parts screwed together. The body of the apparatus is a tube, and upon this slides a second tube having a cross-head held by a strong spiral spring fitted inside the main tube; the sliding tube may be moved by a pair of arms. A set of sounders are provided, and anyone of these can be screwed into the body of the instrument. The diameters of the bases of the sounders vary. In use the apparatus is held vertical by the arms or handles and pressure is put upon it in a downward direction, and this pressure is gradually increased until the sounder enters the surface of the ground to a depth of one millimetre. The position of the sliding tube is then noted on a scale marked in lbs. or kilograms on the main tube. The whole apparatus folds into a small case and only weighs 4½ lbs.

The foundation of the Manchester Ship Canal Grain Elevator. By GERALD GASCOIGNE LYNDE. Vol. cxxxvii, p. 364.

The Manchester Ship Canal Grain Elevator has a storage capacity of 40,000 tons, or 1,500,000 bushels, and is built upon the American principle, the whole of the superstructure being of pitch-pine timber encased with brickwork and tiling.

After the completion of the canal, this low-lying land, which was then outside the canal bank, was between 2 feet and 12 feet below the water level of the canal, and so surrounded by higher ground as to form a pond which in wet weather was flooded.

The Canal Company, who were carrying on a large amount of dredging, decided to fill this land with the sludge brought up by the dredgers. This work was commenced in January, 1896, and the land was filled to a height of about 4 feet above the water level of the canal by the end of May, 1896, when the tipping was

discontinued. A quantity of available small sand and small lumps of sandstone rock was brought up the canal and by means of end-tip wagons was spread over the sludge between 1 foot 6 inches and 2 feet 6 inches thick, bringing up the surface to a height of about 6 feet above water level and nearly level with the canal bank. Before this was completed the foundation for the grain elevator was commenced, the depth of the sludge on the site of the grain elevator being between 14 feet and 18 feet, and it had become of the consistency of butter. When in this condition it was impervious to water as clay puddle, but when mixed with or stirred with water it became a thin black mud.

The foundations were constructed of Portland cement concrete and are of the "gridiron" formation in plan. The longitudinal and cross walls of the gridiron were excavated in trenches until a suitable foundation was reached and then filled with concrete, good ground being known to be obtainable at a reasonable depth. The outside dimensions of the foundations are 454 feet by 86 feet. This is divided into three parts, namely, the eastern end 182 feet long, the centre tower 76 feet long, and the western end 196 feet long. The foundation of the centre tower consists of a solid mass of concrete, the whole area being excavated until good ground was reached. The depth varied between 15 feet and 21 feet, the total unit of excavation for the centre tower being 4,929 yards. The outer boundaries of the eastern and western ends were trenches 6 feet wide.

The ground underlying the sludge consisted of an alluvial deposit, a bed of blue silt 4 feet thick being found at 18 feet below the finished ground level covering a bed of wet running sand 3 feet thick which lay on coarse sand and gravel.

The south trench, being fairly good ground, was sunk with poling boards in the ordinary way.

The centre tower foundation was formed of one large block of concrete 86 feet by 76 feet on plan. It was decided to sink an 8-foot 6-inch trench on each of the four sides so as to form a boundary to the centre tower foundation. These trenches were filled with concrete, and after this had somewhat hardened the middle portion was excavated. Found impossible to sink the trenches of the main foundation by means of poling boards . . . . (Then successfully used sheet piling.)

The concrete in the trenches, etc., was composed of six parts of gravel or broken stone or bricks, two parts of sand and one part of cement, all by measure.

As the concrete in the outside walls was completed the excavation of the enclosed area was commenced. The springing line of the arches was 10 feet below the finished ground level, and it was found that the average level of the bottom of the sludge was 14 feet below ground level, so that there was about 4 feet of sludge below springing level. In consequence of the character of the lower 4 feet, it was thought best to remove the whole and afterwards to refill the space between the concrete walls above the trenches with other material.

On the south side 66 brick columns with stone caps were built in two rows as well as 36 at the centre tower and 24 at the eastern end. These carried the portions of the building which were to be supported on pillars so as to be open beneath for machinery and for loading railway wagons and lorries, the remainder of the building being built from the concrete direct. These brick columns varied between 4 feet and 7 feet in height and about 7 feet square. They were built in cement, the lower portion of common brick and the upper 6 courses of blue brick, and finished with a stone cap 16 inches thick.

It is estimated that the weight on the foundation of the grain elevator is as follows :—

	Tons.
Concrete, brickwork and masonry	27,750
Superstructure and machinery	15,000
Grain	40,000
	82,750

The area of concrete at the foundation is 25,226 cubic feet. The pressure per square foot on the foundation is 3·28 tons. The total cost of the work amounted to £22,000 and occupied 19 weeks.

The engineer for the works, by whom the foundations were designed and under whose superintendence they were carried out, was Mr. W. H. Hunter, Chief Engineer to the Manchester Ship Canal Co.

Well-sinking on the Koyakhai Bridge, Bengal-Nagpur Railway. By GRAVES WILLIAM EVES. Vol. clv, p. 292.

The foundations consist of single wells, 26 feet 6 inches in diameter, sunk 160 feet apart centre to centre. The well curb, for a breadth of 3 feet, splays inwards at an angle of about 45°. The steining is increased inwards by 6-inch offsets every one-foot course till the wall attains a thickness of 6 feet 6 inches, leaving a dredging chamber in the centre 13 feet 6 inches diameter. As much difficulty was experienced in sinking through clay, the offsets were reduced to 3 inches every 1-foot rise and were chamfered to a smooth inward batter of 1 in 4.

The sand of the river was very fine but clean, and was too fine for use in mortar. The clay was dark blue and very hard; brittle when quite dry, but like leather when under water. . . . It was therefore necessary to undercut the steining by other means than by dredging. To do this four methods were available, namely, (1) baling out water, (2) blasting inside the well, (3) blasting in bore holes sunk just outside, and (4) artificially adding weight if no more masonry could be built.

When deciding on the depth to which a well should be sunk skin friction is seldom taken into account. In Indian rivers the depth of scour in a big flood is an unknown quantity. The movement which takes place in the sand makes it necessary to neglect the skin friction for the first 40 to 50 feet at least. The well must be sunk to a depth sufficient to give it stability against overturning, and the depth will generally be found to be greater than that necessary to prevent vertical downward movement.

The value of 2·5 cwt. per square foot for sand was therefore assumed, and the corresponding values for the other unknown quantities from the most reliable data were 12 cwt. per square foot for the friction of the clay and 1 ton 15 cwt. per square foot as the resistance of the clay to being squeezed inwards by the wedge-shaped curb. From these data, and assuming the foregoing values for sand and clay, the value of the vertical reaction was found to be only 6 cwt. per square foot of cross section of the steining.

Whether these figures for skin friction will be useful for determining the depth to which wells ought to be sunk is doubtful. The only way in which wells have been known to fail is by overturning, due to the bed of the river being scoured out.

As the wells are filled with sand from the bottom to within 15 feet at the top the weight of this sand was not taken into account as adding to the stability, since it does not form part of the well but rests on the clay at the bottom quite independently.

The method of sinking the Koyakhai wells decreased the vertical stability of the wells very much. The large blasts of dynamite blew large holes in the underlying subsoil outside the walls, which in many cases were probably not filled up by the sand finally put inside the well. This reduced the skin friction, and when the superstructure was put on the wells the settlement was very great. The south abutment was surrounded with dry stone, solid in front, to a height of about 50 feet. This caused a sinkage of over 4 inches, the bank next the abutment sinking about 18 inches more than it did elsewhere, though consisting of sand.

The Alexandra Dock, Hull. By ARTHUR C. HURTZIG. Proc. Inst. C.E., vol. xcii, p. 144.

The mercantile community of Hull, after several attempts to free themselves from the monopoly enjoyed by the Hill Dock Co. and the North Eastern Railway Co., obtained an Act in 1880 for the construction of the Alexandra Dock on the foreshore of the Humber at Hull in conjunction with the Hull and Barnsley Railway.

As 152 acres of the site of the works, out of a total of 192 acres, were below high-water mark, and extended considerably below low-water mark, the range of spring tides being 22½ feet, an embankment about 40 feet high and 6,000 feet long had to be constructed to reclaim the required foreshore. The actual works comprised a dock of 46½ acres, with 2 miles of dock walls from 52 feet to 62 feet in height, two large graving docks, a lock 550 feet long and 85 feet wide, dredging an entrance channel through the Hebbles Shoal, erecting pumping machinery, etc. The works were commenced in June, 1881, and the dock was opened for traffic in July, 1885.

P. 146. The cofferdam was segmental in form, with a radius of 255½ and 461 feet long, constructed of two rows of piles 6 feet apart, with clay puddle between, the piles being from 50 to 60 feet long and driven about 33 feet into the ground, the mailed piles reaching down to 54 feet below high water.

P. 141. Dock wall foundations.

The dock wall was designed to be founded 12 feet below dock bottom, at which depth it was anticipated that hard clay would be found, and at the first foundation excavated at L (Fig. 1, Plate 2) boulder clay was reached 4 feet above the required level. Proceeding westward, however, the clay varied very much in level, dipping sometimes with a slope of 1 in 4 and rising again 60 to 80 feet further on, the dip being filled with sand (very fine) or silt, which had to be excavated to an extra depth, reaching sometimes 9 feet and sometimes it dipped almost vertically 6 or 8 feet. Eastward the clay was at a good level under the north wall, which could be founded at the intended depth, but at the last 300 feet of the western portion of the wall, the clay, after running 5 to 6 feet below the desired level, dipped suddenly almost vertically, and on excavating the sand to a little lower level water boiled up and the sides soon began to sink. A boring sunk in the trench showed that 12 feet would have to be excavated to reach the clay, which would have risked the collapse of the trench. Accordingly it was decided for the future to enclose the foundations between sheet piling, tongued and grooved, and to found the walls at the designed level, unless clay could be certainly reached a foot deeper. Large chalk was thrown into the enclosed space until a firm bearing was obtained, and then concrete was deposited. The portion of the north wall on this site was finished and backed up for nearly three year

before the water was let into the dock, and showed no sign of settlement or displacement.

The site for the graving docks was enclosed with grooved and tongued sheet piling driven into the clay, and the work commenced at the designed levels irrespective of the nature of the stratum reached. The masonry of No. 1 Graving Dock was laid on a very dry foundation of clay, peat and silt. No. 2 being deeper was carried to a lower level, and the masonry was laid upon a bed of 3-foot Portland cement concrete.

The chimney shafts of the engine-houses and the accumulator towers were built on a foundation of Memel or pitch-pine piles 30 feet long covered with a double timber platform. The piles were driven to a resistance of half an inch under a 1-ton monkey falling 10 feet, and were subjected to a maximum load of 15½ tons per pile, which, according to diagrams made by the Author, gave a factor of safety of four.

(General description follows, but no pressures per square foot, etc., are mentioned.)

Cofferdam of the Centre Pier of the Arthur Kill Bridge. By A. P. Boller. Trans. Am. Soc. C.E., vol. xxvii, p. 475.

The centre pier of this bridge, uniting the Jersey shore with Staten Island at Elizabethport, is founded upon the red sandstone of the district, and was built within a coffer dam of somewhat novel construction with a view of avoiding all interior bracing, which interferes greatly with rapid and economical building of masonry.

The physical conditions at the site of this pier were a nearly level rock bottom (the sandstone being in its natural bed with not over 10 inches pitch to the eastward in the width of the dam) overlaid with about 2 feet of clay under some 18 inches of sand and mud and a depth of water over the rock of 28 feet at high tide. The plan of dam adopted was a double-walled polygon of twelve sides, the walls being 4 feet apart in the clear within which the puddle was placed. The inscribed circle of the inner wall measuring the free working space was 42 feet 6 inches in diameter. The walls were built up of square hemlock timbers as shown, halved into each other at their intersection and proportioned under the consideration of a horizontal-polygonal ring, subject to a uniform load of water due to the head at any point. The separate walls were tied together by bolts and round struts.

Outside radius of dam = 28 feet; inside = 22 feet.

Between each course of timber and at the scarfing of the joints a line of cotton wicking was introduced which by swelling would aid tightness of the walls and prevent the puddle seeking, under a strong head of water, a vent caused by unevenness of timber, which events proved to be a wise precaution.

Previous to launching the dam the site of the pier had been prepared by dredging the rock bare and settling in place the crib blocks on either side, constituting part of the permanent fenders, which are all crib work.

Before pumping out the bottom was prepared by depositing under water 4 feet of concrete all over the dam. After allowing the concrete to harden for a week the dam was pumped out in a few hours for the masons to start work, and a beautifully tight dam it was, with one exception, and that was a very small area about 5 feet from one of the walls where the divers had omitted to properly cover the bottom with concrete, and quite a lively spring spouted up from the bottom.

Diagram p. 476 shows pier No. 3, presumably the above caisson, resting on sandstone nearly sunk through (from rock up) 2 feet 6 inches of clay and 1 foot 2 inches of sand with 28 feet head bottom to high water.

The New Bridge of the London Chatham and Dover Railway Company over the Thames at Blackfriars. By GEORGE EDWARD W. CRUTTWELL. Proc. Inst. C.E., vol. ci, p. 25.

General Description.—The new St. Paul's Station in Queen Victoria Street was designed to provide additional accommodation, long needed, on the City side of the river for the increasing traffic on the London Chatham and Dover Railway.

Within the station are three terminal lines of way, and on the west side, adjoining the previously existing lines of the same railway, are two through lines, which form a junction with the old lines a little to the south of Ludgate Hill Station. The new lines are carried across the river by the new Blackfriars Bridge, which forms the subject of this Paper.

The bridge is designed to carry in all seven lines of railway, and has a clear width of 81 feet between the parapets, but this width is increased to 123 feet at the northernmost span, where it was necessary to provide space for the platforms as well as for the lines of the way.

In consequence of the great width, and the numerous cross-over roads upon the bridge, it was impossible to employ a construction like that of the older bridge, consisting of main girders rising above the rail level, and therefore a system consisting of arched ribs beneath the rails was adopted.

The bridge has 5 spans, the shore span on the Surrey side being 183 feet, the centre span and the Middlesex shore span 185 feet each, and the second and fourth spans 175 feet each; the lengths of the three middle spans correspond with those of the old bridge alongside.

Abutments.—The cofferdams enclosing the abutments were each composed of a single row of sheet piles of sawn pitch pine driven to an average depth of 25 feet below the foreshore, or about 7 feet below the foundations of the arches. The cofferdams were set forward some 27 feet in advance of the foundations in order to utilise them for the construction of wharves.

On the outside of each cofferdam a trench was dug 2½ feet wide by 3 feet deep and filled with clay puddle. The piles being of sawn timber, caulking was unnecessary except in a few places where leaks appeared on the occasion of the closing of the dam. The leaks were speedily stopped and the excavations for the abutments were kept dry by means of a 10-inch pump worked by a 6-HP. engine.

The excavation for the abutment on the Middlesex side of the river was carried down throughout its whole area to a depth of 15 feet below Trinity high water, and below this the excavation was carried 10 feet deeper in bays about 12 feet in width by 37½ feet long, the latter dimensions corresponding with the width of the foundations. Each bay was filled with concrete before the excavation of the adjoining bay was commenced. The bed of the foundations, at a level of 25 feet below Trinity high water, is about 1 foot beneath the surface of the ballast overlying the London clay. The surface of the clay is about 15 feet beneath the foundations of the abutment.

Piers.—The foundations of piers 1, 2 and 3 (counting from the Surrey side) were each sunk within three rectangular caissons, spaced about 6 feet apart, whilst for the long pier on the Middlesex side four caissons were required.

Immediately above low-water level arches were turned between the separate portions of the piers built within the adjoining caissons; above the archways the piers are continuous.

The pier foundations are all carried down to a depth of 46 feet below Trinity high water, where they rest in the London clay, the depth below the river bed varying from 16½ to 23½ feet.

The Caissons.—The caissons are rectangular, varying in dimensions from 30 feet × 32 feet to 30 feet × 26 feet. They consist of a single skin of wrought-iron plate ½ inch thick for the bottom 7 feet and diminishing to $\frac{5}{16}$ inch at the level of high water. The lower portion, or permanent caisson, in each case is 21 feet in height, and above this a temporary caisson was carried to a further height of 28 feet, this portion being bolted to the permanent caisson and removed after the building of the pier inside was sufficiently advanced.

Excavating inside the Caissons.—The caissons were weighted with kentledge to facilitate the sinking, and under this load the cutting edge penetrated at first below the excavation, but as the sinking progressed and the friction increased it became necessary to excavate down to the cutting edge, and in some cases it was with difficulty that the caissons were forced down to their full depth. This was especially so in the case of the caissons of pier No. 4, which had to be sunk through thick layers of septaria embedded in the London clay.

To aid the descent of this pier hydraulic presses were applied, the purchase being obtained against the undersides of inverted trussed beams laid across the caissons and secured to the piles of the staging surrounding the pier. When the presses at each corner of one of these caissons were exerting a pressure of 30 tons each, or 120 tons together, the total load on the caisson amounted to 390 tons, the hydraulic pressure being aided by the weight of the caisson, including timbering, etc., which was estimated at 143 tons, and also by the weight of the kentledge amounting to 127 tons.

The least pressure exerted was in the case of one of the caissons of pier No. 2, when the caissons, including timber, etc., weighed 108 tons, and the kentledge 117 tons, or a total of 225 tons.

The average depth through which the caissons were lowered whilst the grabbing and divers' work was in progress amounted to about 4 inches daily, but when the pumping out and excavating was commenced the average descent was increased to about 15 inches, the maximum in any single day having been 2 feet 9 inches.

Construction of the Piers.—The foundations of the piers inside the caissons consisted, for a height of 17 feet above the bottom, of solid cement concrete in the proportion of six parts of concrete to one part of cement . . . . At a level of 4 feet below the top of the permanent caisson the brickwork was commenced.

At the level of the top of the permanent caisson, or 25 feet below Trinity high water, a set-off of brick occurs, and at this level the granite facing begins.

P. 36. The load upon the foundations of the piers amounts to 4¾ tons per square foot, with the maximum possible load upon the bridge. There was no settlement under the Board of Trade tests. (The lines of way on each span were loaded, two at a time, with locomotives extending the length of a complete span.)

The joint engineers for the work were Mr. William Mills and Messrs. J. Wolfe Barry and H. M. Brunel; the Author was the Resident Engineer.

Plate 5, Fig. 5, shows a sectional plan with outside dimensions of 98 feet × 30 feet, instead of 90, as stated in Mr. Cruttwell's letter.

Address of Sir George B. Bruce, President. Proc. Inst. C.E., vol. cxi, p. 7, 1888.

I may refer to a few of the bridges founded by means of cylinder, or caissons.

The new Tay Viaduct, of which Mr. Barlow, Past-President, is the engineer, has main spans of 245 feet, each pier carrying which is formed of two iron cylinders 23 feet in diameter filled with brickwork and concrete and sunk to depths varying from 20 feet to 30 feet into and resting upon sand, the depth of water at high tide being 23 feet. The weight borne by each superficial foot in the cylinders, including rolling load, is estimated at 3 tons.

The "Empress" Bridge over the Sutlej in India has spans of 250 feet, and each pier is formed of three brick wells of 19 feet outside diameter, and they are sunk on an average 110 feet into the bed of the river.

The bridge over the Ganges at Benares, with spans of 335 feet, has its piers, composed of single iron caissons of oval shape 65 feet long by 28 feet broad, lined with brickwork and filled with concrete. These are sunk to a depth of about 100 feet.

The bridge over the River Hooghly, 30 feet above Calcutta, of which Sir Alexander Rendel and Sir Bradford Leslie were engineers, has a single cantilever carried on two piers, which were founded by means of wrought-iron caissons 66 feet long by 25 feet wide with semicircular ends. These were sunk to a depth of 73 feet into the bed of the river and 108 feet below the lowest water level. Each caisson had in it three compartments through which the earth was excavated by means of vertical annular boring shafts driven by steam power and armed at the bottom with radial cutters which excavated circular holes of 10 feet to 15 feet. The material was removed by a current of water flowing up the hollow shaft and over a syphon into the river. The flow was maintained by pumping water into the excavating chambers and keeping it at a higher level than the water in the river.

In the foundations of the Forth Bridge the caissons are of very large dimeusions, being 70 feet in diameter, and the deepest reached depths varying from 71 feet to 89 feet below high water and from 39 to 43 feet into the bed of the Forth. In the cases of the other bridges referred to (excepting over the Hooghly), the caissons or cylinders were sunk by having the material excavated from the inside by means of grabs and other tools working in the open, but in the case of the Forth the pneumatic process was in the main adopted. The men worked in compressed air in a chamber 7 feet high, occupying the whole of the bottom surface of the caisson, the chamber being filled with concrete after it had reached its proper depth.

One of the most remarkable instances of the sinking of foundations by means of iron caissons was exhibited in the erection of a graving dock at Toulon. Here the caisson was 472 feet long by 134 feet wide and 62 feet deep and embraced the entire dock, which was built of masonry. The excavation necessary for sinking it was carried on, as in the case of the caissons for the Forth Bridge, by the use of a compressed air chamber in the bottom of the caisson.

The Campen Lighthouse and the Illumination of the Lower Ems. By C. RIENSBERG. Inst. C.E., vol. ciii, p. 435.

Emden, formerly a Hanoverian port, was dredged by the Prussian Government to a depth of 23 feet and harbour lighted by 6 lighthouses. The most important of these structures is near Campen.

The foundations are carried through soft alluvial soil to a depth of about 40 feet, where firm hard soil is reached. The diameter of the pier foundations is 19 feet, and that for the shaft 14 feet. The outer facing is of masonry, with concrete filling. Each pier is held by an anchor plate built in at a depth of 26 feet 3 inches with four holes 4 feet in diameter, which is calculated for a collective strain of 135 tons in the severest hurricane.

The calculations for the strength of the structure are based upon the extreme assumption that every part will be exposed to the full force of hurricane pressure, and the maximum strains in various parts of the bracing and piers do not exceed 2·6 tons per square inch in compression and 4·45 tons in tension.

The New Chittravati Bridge. By EDWARD W. STONEY. Proc. Inst. C.E., vol. ciii, p. 135.

At a distance of 212¾ miles from Madras the main line of the Madras Railway crosses the Chittravati River. The new Chittravati bridge has a total length 2,680 feet consisting of 19 spans of 140 feet each from centre to centre of piers.

At the south abutment rock lies at a depth of 18 feet below the present bed and dips gradually to a maximum depth of 80 feet at pier No. 17. Above the rock the deposits consist of varied and irregular strata of sand, gravel, clay and large trap boulders, while mixed with the sand were found water-worn pebbles and large fragments of rock, some sharp and others rounded.

The Chittravati River rises 80 miles above the bridge, and in this distance drains an area of 2,400 square miles. Its fall at the bridge is at the rate of 8 feet per mile, and its mean velocity and discharge during the flood of 1874 were calculated to have been 8·46 feet per second and 114·625 cubic feet per second respectively, and it is believed that the sandy bed of the river was then scoured to a depth of 15 or 20 feet. As a rule the river remains practically dry for about 9 months in the year, although the water level never falls lower than about 3 feet below the surface of the sand.

P. 137. *North Abutment.*—3–12 foot cast-iron cylinders were used in the centre under the body of the abutment, with four brick wells, two under each wing wall.

In order to avoid the trouble, delay, and expense of loading these cylinders on the top with rails, they were sunk by weighting them with an internal lining of masonry set in cement mortar. The ring of masonry was carried on an annular plate of cast iron designed for this purpose and fixed between two lengths of cylinder.

The weight of masonry ring before immersion was about 4 tons per lineal foot of the cylinder.

Priestman's grabs were used for dredging out the material, the sinking being continued until the cylinders reached the bed of boulders at a depth of 60 feet below the surface. The excavation was then carried on by divers until the cylinders were sunk to a bed levelled in the rock at a depth of 66 to 68 feet.

The brick wells were built on wrought-iron curbs. They were carried down to the boulders and were bedded by divers at a depth of 60 to 63 feet below the surface. Considerable trouble was experienced in sinking them, as they were firmly held by the top stratum of stiff clay, which was 27 feet thick. It sometimes happened that a hole had to be dredged in the centre 14 to 20 feet below the cutting edge before the well could be made to sink, and in such cases the outside sand would rush in, forming a crater all round. In consequence of these slips some of the wells were drawn out of plumb, and one under the west newel canted outwards so that when sunk to the proper depth it came in the way of the next well, which struck upon its curb during the progress of sinking and could not be got any deeper. This incident illustrates the necessity of leaving ample room between any contiguous cylinders which have to be sunk to considerable depths so as to prevent their coming in contact if they get slightly out of plumb. The tops of the walls and cylinders are adjusted at a level 6 feet below the river bed and are united by arches on which the superstructure of the abutment is carried up.

Cylinder Piers (p. 138).—Each of the eighteen river piers consists of a pair of cast-iron cylinders, placed 18 feet apart from centre to centre, and braced together at the top by a deep and massive box of plate and angle iron.

The cylinders have a diameter of 12 feet throughout the lower portion to within 9 feet of the river bed, at which point a conical tapering length is inserted reducing the diameter to 9 feet.

All the piers, with the exception of pier No. 11, were bedded on the solid rock, the bottom being dressed level, etc.

As far as pier No. 9 the sinking and bedding of the cylinders was comparatively easy, but beyond this point all the piers gave considerable trouble, especially Nos. 10, 11, 12, 13 and 14, as the bedrock was here covered by a depth of from 7 feet to 22 feet of boulders, the largest taken out unbroken being 5 feet long by 11 feet in girth and containing about 45 cubic feet. When large boulders were met under the cutting edge it was a difficult matter to remove them safely. If they were pulled into the cylinders it generally happened that a blow of sand would follow; and when the projections were cut off by blasting the cylinders were sometimes cracked by the dynamite, although the charges used were small, while the drilling of the holes by divers was always a tedious operation.

P. 143. Taking the average of all the cylinders a depth of 18 feet of concrete at the bottom was required to staunch them, and the top of this was at an average depth of 30 feet below water level in the river; but this dimension varied greatly in the different piers, the maximum depth being 50 feet below water with 3 feet of sealing.

P. 144. Above the concrete the cylinders were filled to within 3 feet of the summit with hammer-dressed masonry of fine flat-bedded limestone.

The bridge was designed by Messrs. Hawkshaw, Son and Hayter, by whom the ironwork was sent from England. The laying out of the work and its entire management and supervision were entrusted to the Author.

TABLE I.—LOADING OF CYLINDERS SUNK BY THE PNEUMATIC PROCESS, AND FRICTIONAL RESISTANCE IN SINKING AS FOUND BY NINE EXPERIMENTS IN THE CYLINDERS.

—	Pier 11. (Left Cylinder.)			Pier 11. (Right Cylinder.)					Pier 13.
	A	B	C	A	B	C	D	E	
	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.
Imbedded depth of cylinders :—									
Sand	33 2	33 2	33 2	33 2	33 2	33 2	33 2	33 2	33 8
Clay	10 2	10 2	10 2	10 2	10 2	10 2	10 2	10 2	10 0
Sand and clay	7 3	7 3	7 3	0 8	4 6	7 3	7 3	7 3	1 0
Clay and boulders	5 8	7 5	9 1	..	..	7 11	7 11	12 5	2 4
Total	56 3	58 0	59 8	44 0	47 10	58 6	58 6	63 0	52 0
	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.
Area of imbedded cylinder surface, sq. ft.	2,165	2,232	2,296	1,693	1,841	2,251	2,251	2,426	1,960
	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
Load employed, weight of cylinders in tons	78·28	..	..	70·22	70·23	85·12	78·27	78·27	70·83
Load employed, weight of rails, tons .	222·22	..	..	173·33	248·88	222·22	222·22	222·22	229·70
Load employed, other extraneous load in tons	10·30	..	..	4·74	4·74	4·74	4·74	10·30	10·30
Total load	310·80	310·80	310·80	248·30	323·85	312·08	305·23	310·79	310·83
Lifting force due to air pressure observed	40·32	44·64	44·64	248·30	323·85	312·08	305·23	310·79	49·10
Net sinking force	270·48	266·16	266·16	248·30	323·85	312·08	305·23	310·80	261·73
	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.	Cwts.
Horizontal resistance per square foot of imbedded cylinder surface in cwts. .	2·50	2·38	2·32	2·93	3·52	2·77	2·71	2·56	2·67

Appendix B.—The data obtained in sinking cylinders by the pneumatic apparatus ought to give very accurate results as the interior of the cylinder was cleared and under-cut to a depth of 3 feet below the cutting edge. The air was then allowed to leak off and the pressure at which the cylinder sank was noted. Therefore at the moment when the cylinder began to move the external load just overcame the cylinder surface friction and residual air pressure.

Surface Friction of Cylinders as deduced from Loads actually used.	Cwts. per Square Foot.		
	Mean.	Maximum.	Minimum.
Average of thirty-six cylinders sunk to depths varying from 10 feet to 27 feet and averaging 19 feet under their own weight only, which varied from 25½ tons to 33 tons, and averaged 31 tons . .	0·85	1·33	0·63
Average of 100 observations in sinking thirty-six cylinders at depths varying from 17 feet to 64 feet, under a load of rails varying from 31 tons to 249 tons and averaging 132 tons besides the weight of the cylinders	2·13	4·08	1·29
Average of nine observations in sinking three cylinders by pneumatic process at depths varying from 44 feet to 63 feet, as shown in detail in Table I	2·71	3·52	2·32

The River Spans of the Cincinnati and Covington Elevator Railway, Transfer and Bridge Co. By Wm. H. Burr. Trans. Am. Soc. C.E., vol. xxiii, p. 47.

This structure crosses the Ohio River at Cincinnati, Ohio, and with its approaches forms a part of the Chesapeake and Ohio R.R. system. It acquires its interest as a piece of engineering chiefly from the magnitude of the individual spans of which it is composed. There were no special engineering difficulties to be overcome either in the substructure or superstructure, but the central span of the three, 550 feet long between centres of piers and 84 feet deep between centres of chords, is the greatest simple non-continuous truss span yet constructed. The two spans which flank the centre or main channel span are 490 feet each between pier centres, with centre depths of 75 feet; and the fact that all the spans carry a double track railway with two roadways and two sidewalks renders them also the heaviest non-continuous trusses which have yet been built either in this country or in Europe.

Substructure.—The shore piers of the two 490-foot spans rest on piles, capped transversely of pier, with 12 inches by 12 inches white oak timbers, which in turn carry longitudinally of pier 9 lines of the same 12 inches by 12 inches timbers. These latter carry a solid 12-inch white oak floor or platform about 72 inches by 36 feet, on which the masonry is placed. The piles are placed 4 feet apart, centres in both directions. They are white oak sticks driven to refusal 30 feet to 42 feet into the clay and gravel of the banks. There are five bottom courses of masonry, each 27 inches thick and each stepped off 12 inches. The masonry of the main body of the pier surmounts these bottom courses with the batten and dimensions shown on Plate XII.

The 24-inch subcoping courses on all the piers are of Kentucky freestone, while all the 24-inch coping courses are volitic limestone from Salem, Indiana. This latter is a very compact stone and offers a compressive rest of about 12,000 lbs. per square inch; its ratio of absorption does not exceed 2 per cent. of its weight. The belting courses are of a very superior sandstone from the interior of Kentucky, known as Kentucky freestone. It possesses a compressive resistance of about 15,000 lbs. per square inch and a ratio of absorption of 3 per cent. The Kentucky shore pier was built of this freestone throughout, while the Ohio shore

pier is entirely built of Ohio river freestone. The two river piers are faced with Greensburgh limestone, and both are backed with Ohio River freestone from top of caisson to belting course. Above the latter the same backing was used in one river pier and the Kentucky freestone in the other.

The two river piers, one at each end of the 550-foot span, rest on pneumatic caissons 81 feet 3 inches by 34 feet 10 inches in plan at cutting edges. The batten of the caisson is 1 foot in 15 feet. The walls of the working chamber for a distance of 6 feet below the roof are 4 feet thick and composed of three shells of 12-inch by 12-inch sticks with four of 3-inch sheathing alternately. The outer shell of 12-inch sticks is carried down 2 feet below the interior and 1 foot below the centre one ; the former carries at its lower extremity a 6-inch by 9-inch piece of oak chamfered to form the cutting edge. The distance from base of cutting edge on shore to the roof of the caisson is 8 feet 9 inches, etc. The roof is formed by seven solid transverse and longitudinal layers of 12-inch by 12-inch pine sticks. Above the caisson is constructed the cribwork or grillage formed of alternate layers of four longitudinal and eight transverse 12-inch by 12-inch pine sticks with interstices, forming by far the larger part of the mass, filled with the best of concrete. This cribwork consists of thirty-five layers in the Ohio caisson and thirty-four layers in the Kentucky one, above which comes the masonry of the pier proper. The top of the cribwork is about 30 feet by 76 feet, and the distance from the top of the cribwork to the cutting edge is 52 feet 5 inches for the Ohio, and 51 feet 3 inches for the Kentucky caisson.

P. 58. As already stated, the work of sinking the caissons was begun July 1st and continued without serious interruptions until October 12, when the depth of cutting edge below low water was 52 feet 9 inches, with air pressure 23 lbs. per square inch. At this depth the pressure on the longitudinal walls was so great as to show some bending of the middle transverse bracing, and as bed rock was found at a depth of 1 foot 9 inches only below the cutting edge at this stage of the work, it was deemed advisable to attempt no further sinking of the caisson. Excavation was then made from the entire cutting edge to bedrock and the whole carefully sealed with concrete. This was done in 10-foot sections, etc., the cement used being Alsen's German.

The entire working chamber was then thoroughly cleaned out with great care. This soapstone ledge and the first two thin layers of limestone which overlaid the bedrock of limestone were entirely removed.

The pneumatic work of the Ohio caisson was finished on October 31st at 4 P.M., the masonry of the pier being 22½ feet high. The caisson rests on bedrock, and its position is precisely right. It was originally placed 12 inches upstream with the anticipation of its being drifted that much downstream before work was completed, and the expectation was exactly realized.

Résumé of Ohio Caisson :—

Time of sinking 104 days, or 6 inches per day on average from low water.

Total time occupied, 133 days from time caisson was sunk until completed.

5,276·6 cubic yards displacement.

Cutting edge 52 feet 9 inches below low water.

Bedrock 54 feet 6 inches below low water.

Bed of river to cutting edge 45 feet 9 inches.

The Kentucky Caisson (p. 59).—The material passed through was sand, gravel and large boulders, being apparently through the original bed of the river after getting down some 20 feet.

Total working days from time of location, 120 ; average per day sinking,

August 5th to September 27th, viz., distance bedrock to low water 53·5 feet; average 0·575 foot per day.

Distance bedrock to bed of river, 42 feet; average 0·451 foot per day.

153,383·73 cubic feet displacement.

The Kentucky caisson remained in its first position about 12 inches upstream. Piers are of limestone, freestone and oolitic stone.

The total weights, including timber in substructure, concrete, etc., spans, timber of same and maximum moving loads on the various abutments and river piers, and loads carried per pile on the abutment piers, and per square foot at bottom of caissons for the two river piers, are as follows:—

	Lbs.
Ohio abutment pier, total weight	13,202,324
Load per pile	77,200
Kentucky abutment pier, total weight	13,890,224
Load per pile	81,200
Ohio River pier, total weight	36,719,285
Total load per square foot	13,000
Kentucky River pier, total weight	36,922,285
Total load per square foot	13,047

The above total weights sustained by the two river piers are the actual total loads less the buoyant effect of the displacement, the volume of which is given in preceding data.

The pneumatic portion of the substructure, including all caisson and cribwork, was performed by Messrs. Sooy Smith and Company during 1887 and 1888 in their usual efficient and successful manner.

The Cantilever Highway Bridge at Cincinnati. By GUSTAVE KAUFMAN and F. C. OSBORN. Trans. Am. Soc. C.E., vol. xxvii, 1892, p. 173.

During the years 1890 and 1891 the cantilever highway bridge described in this article was built across the Ohio River between the cities of Cincinnati, Ohio and Newport, Kentucky. The roadway is 24 feet wide in the clear, with two sidewalks each 7 feet wide. The total length of the structure is 2,966 feet. The main engineering feature is the cantilever span, 520 feet from centre to centre of piers.

The site of the new bridge is very favourable for economical construction from the fact that a peculiar limestone formation extends across the river at this point. The top of this formation is an irregular triangle in shape, the base of which is on the Kentucky side and the apex on the Cincinnati side. On the Kentucky side at extreme low water this formation is exposed. The base of the triangle is about 1,400 feet long, and extends from the mouth of the Licking River to a point midway between the bridge under discussion and the Louisville River to a point midway between the bridge under discussion and the Louisville and Nashville Bridge. The top of the formation maintains the level of extreme low water about two-thirds the distance across the river, then it drops suddenly, and for the balance of the distance across the river the top of the rock is from 5 to 7 feet below low water. The site of other bridges built across the Ohio River at Cincinnati were by no means so favourable, and it was necessary to go to considerable depths to obtain suitable foundations for their piers, notably the Chesapeake and Ohio Railway Bridge, where rock was found about 52 feet below low water.

P. 179. The superstructure is supported by 2 abutments, 28 pedestals and 9 piers. The abutments and ramp walls are built of second class masonry and entirely of Ohio River freestone, except the coping, which is of Barea sandstone. All the pedestal piers are built of first class masonry.

Piers Nos. 1, 2, 3 and 9 are similar in all respects, except as to size and height, and are all founded on piles driven to a firm resistance from short blows of a hammer weighing 4,000 lbs. The foundation beds were from 7 to 10 feet deep, and after sawing the piles off 18 inches above the bottom of the pits, concrete was put in varying in thickness from 3 to 4½ feet, thoroughly embedding the piles in a plastic mass upon which the foundations and footing courses were started. These piers were built entirely of Ohio freestone, except the coping, which was Bedford oolitic limestone. They are rectangular in plan throughout their height, battering ½ inch to the foot, and as they stand above the average high water no difficulties were encountered in their construction.

Pier No. 4 rests upon 150 piles driven to solid rock, having heavy cast-iron shoes, the points of which were seated in the rock by repeated light blows from the hammer. They were cut off 18 inches above the bottom of the foundation bed and their heads were imbedded in concrete 3 feet 6 inches thick. Upon this the foundation footing courses, four in number, were laid.

Piers Nos. 5, 6 and 7 are similar in construction and are located in the river. Piers Nos. 6 and 7 are founded on solid rock, and their foundations were put in without difficulty by the use of single wall cofferdams. The solid rock bed in the river at this point has no deposit upon it, and in landing the cofferdams it was necessary first to sink a crib composed of timbers and stone above the pier site in order to hold the cofferdam in place. The cofferdams for piers 5, 6, and 7 were all alike in construction, being rectangular in plan and 30 × 70 feet in size outside to outside. The walls were built of horizontal courses of 12 inches × 12-inches timber, bolted together for a height of 6 feet and above, though the walls were of 6-inch × 4-inch stuff.

The bottom edges of the walls were padded with cotton waste 6 inches thick, held in place by cotton ducking. Considerable difficulty was experienced in pumping out this cofferdam, the cotton ducking having been torn out in a number of places in launching, causing leaks which were finally stopped by throwing in bags of sand.

As winter was rapidly approaching it became obvious that if the work was to be finished approximately on time it was absolutely necessary that the river work be completed first. The condition of pier No. 5 and the condition of the river made it clear that some radical move had to be made. It was determined on September 15th to use the pneumatic process in founding pier No. 5, notwithstanding the fact that bedrock was only about 7 feet below low water.

The caisson was 12 feet high from the shoe to the top of the deck, with a cofferdam about 24 feet high, so that the work could be prosecuted in a 24-foot to 26-foot stage of water after the caisson was landed on the rock bottom of the river.

The caisson in this foundation was of the Morison type, except that the iron shoe was omitted. The sinking was accomplished without accident or injury to any of the men engaged on it, and required 720 hours' actual working time to penetrate 8 feet into the solid rock, or an average of 3·2 inches for each 24 hours. The rock penetrated consisted of ledges of fairly hard shaly formation alternating with their ledges of hard fossiliferous limestone. When first struck it was not well adapted to make a good foundation, and in order to get the deck of the

caisson 3 feet below extreme low water it was necessary to penetrate the rock 8 feet.

P. 195. Piers 4, 5, 6, 7 and 8 were also rectangular in shape in a horizontal section from the top of coping down to the elevation of high water. From high water down they have a semicircular nosing at each end, as shown.

The general dimensions for each pier are approximately as follows :—

	Coping. Feet.	Height. Feet.		Total Height.
Pier	4—11 × 36	29	41	70
,,	5—12 × 36	34	79	113
,,	6—12 × 36	34	71	105
,,	7—11 × 36	29	70	99
,,	8— 9 × 32	21	70	91

P. 218. The caisson to be used in building the foundation for pier No. 5 was 22 feet × 58 feet at top, 12 feet high over all, and has a batter of ½ inch per foot from a point 1 foot below the top to the cutting edge, thus making the bottom 22 feet 11 inches × 58 feet 11 inches. Timber, oak.

The construction of the Dufferin Bridge over the Ganges at Benares. By F. T. G. Walton. Min. of Proc. Inst. C.E., vol. ci, p. 13.

The bridge over the River Ganges at Benares was constructed by the Oudh and Rohilkhand Railway Co. to connect the line, already built from Benares to Lucknow, with the East Indian Railway. This is the fourth bridge over the Ganges which has been erected by the same company, and by means of it a more direct route is supplied between Bengal and the provinces of Oudh and Rohilkhand. By it also a duplicate and more direct link is formed between Bengal and the Punjab.

The bridge was commenced on the 19th of January, 1881, was tested on the 24th of September, 1887, and formally opened in December, 1887.

The site is immediately below the tower of B, which has existed for many centuries on the left bank of the river. The town is built upon a thick bed of clay against which the river impinges, and is thus restrained into a channel which is probably narrower and deeper at this point than at any other part of its course. The clay projects below the river bed as far as the middle of the present channel, and upon it the left abutment and the three adjoining piers of the Dufferin Bridge are founded. The remainder of the site is entirely of sand, which extends for many feet below the river bed.

At the site of the bridge the original level of the left bank appears to have been just above the highest flood level, but it has been artificially raised between 30 and 40 feet by the debris of the old city, which formerly extended considerably below this point. This debris consists chiefly of clay, bricks, and tiles used in the constant rebuilding of houses, and is almost as solid as the original bank.

The right bank is about 7 feet below highest flood level and is subject to nundation for a distance of 5 or 6 miles inland.

The Ganges is joined by the Jumna at Allahabad about 100 miles above Benares; and the water level below the junction is chiefly affected by the fluctuations of the Jumna, which, being fed by rapidly collecting streams in the

highlands of Central India, often bring down sudden and extensive floods during periods of heavy rainfall in the collecting area. The rise of water from lowest level to highest flood in the Jumna is 49·25 feet at Allahabad, and at Benares 50 feet; whereas at Cawnpore, situated on the Ganges, 120 miles above the junction of the Jumna, the total rise is only 19 feet.

Floods, however, only occur during the rainy season, from June to October; and with the exception of slight rises of a few feet in January and February, the stream at Benares may be depended upon to fall gradually from the cessation of the rains in October until their recommencement in June. During the dry season the current gradually lessens until its velocity is not more than a mile an hour and the channel then silts up considerably; but during the rains the velocity of the current is occasionally 15 miles an hour and the bed is sometimes scoured to a depth of 70 feet below low-water level, which gives a possible depth in full flood of 120 feet. Hence, in undertaking works below high-flood level, it is necessary to fix upon a programme for each season and to work during that season at such high pressure as will insure the programme being so far carried out that all work in the river bed shall be secure from the effect of the floods.

The greatest depth of the river in the dry season, when the works were commenced, was 37 feet, and the depth during the highest flood was 100 feet; since the piers have been completed these depths have increased to 65 feet and 120 feet.

The bridge consists of seven spans of 356 feet and nine of 114 feet from centre to centre, and is built for a single line of 5 feet 6 inches gauge. The railway is carried between the main girders of the large spans, and on top of the girders of the smaller spans. The height from low water to rail level is 79 feet and the clearance under the main spans at highest flood is 25 feet.

River Piers (p. 16).—The piers carrying the 356-foot spans are all founded on a single well of elliptical shape 65 feet across the major axis and 28 feet across the minor axis. Five of these wells were sunk in water varying in depth from 7 to 20 feet. At piers Nos. 1 and 5, where there was only a depth of 7 and 12 feet of water respectively, the wells were started on earth banks thrown into the river above water level.

For each of these large piers iron caissons were provided having three internal excavating chambers divided by cross walls. Three of these caissons were 10 feet in height, two of 26 feet, one of 42 feet and one of 50 feet, to suit the varying depth of water in which the piers had to be built. They were constructed with the inner shell at an angle of 45° up to a height of 6 feet, thus forming a cutting edge, which was strengthened by a cast-steel shoe 1½ inch thick. The cross walls in the caissons were not carried down as far as their base, but commenced at a height of 6 feet, they also being provided with cutting edges. The caissons were constructed with iron plates ½ inch thick up to 6 feet in height, and beyond that ⅜ inch thick, the inner and outer shells being connected by bracings both horizontal and diagonal.

The caissons of piers Nos. 1, 6 and 7, which were all built on the dry bank of the river or on artificial islands, were sunk by hand excavation as far as the water level. The sinking was in general continued below water level by dredges, unless, as was the case in piers 1, 2 and 3, a clay stratum, met with at varying depths, necessitated further excavation by hand. The caissons of piers 2, 3, 4 and 5 were constructed on and lowered from the pontoons, and were sunk by means of Bruce and Batho's dredgers working on staging erected on the pontoons.

The pontoons were of iron, four in number, all 200 feet long by 25 feet wide and 10 feet deep.

A pair of pontoons having been moored approximately in position for commencing one of the piers, the first 10 feet of caisson was built upon a false flooring of timber, and when caulked was raised from the floor by four chains passing under the cutting edge and secured by shackles and eyes riveted to the inner side of the caisson. The floor was then removed, and the suspended caisson was gradually lowered as its construction proceeded, the weight being augmented as required by the concrete and brick filling until the cutting edge approached the bed of the river. The caisson was then carefully adjusted in position, and when grounded the chains were released by divers, and the brickwork being then carried up to about 12 feet above water level, the pier was ready for sinking (p. 17).

Pier No. 6 was 4 feet 9 inches out of the perpendicular when 109 feet in the ground, and No. 7 was 5 feet 3 inches out when 135 feet in the ground.

The steps taken to right these wells were as follows: the steam hoists were removed to the side to which the wells required to be drawn; a weight of about 1,000 tons in rails was placed on the top of the well in such a manner as to bring the greatest weight to bear on the high side. The earth outside the well was then excavated on the side toward which the well was leaning down to water level, leaving the slope of the excavation 1 to 1. This slope was covered with wooden sleepers at intervals, and on these, in the direction of the slope, were laid rails 6 feet apart. Again, on the rails were laid sleepers close together, and the whole excavation was then filled in with bricks, forming a large wedge-shaped mass pressing against the side of the well continuously as it sank. In this way the wells were drawn over toward the vertical. No. 6, which was 4 feet 9 inches out of the vertical, was brought over that distance while sinking only 5 feet 8 inches, and No. 7, which was 5 feet 3 inches out of the vertical, was corrected in sinking 20 feet 6 inches further.

In sinking with the large diggers the usual progress was 2 feet in 24 hours, the diggers being worked at night by the aid of electric light and the masonry proceeding in the day; but the rate of progress varied according to the nature of the ground.

In the case of pier No. 4 the average rate of sinking amounted to 0·86 foot per day, and the average depth sunk per working day of the diggers was 1·43 feet; but the work was seriously delayed by an accident that occurred on 17th April, 1883.

The well had been founded on a caisson 26 feet in height, and at the time had been built up to a height of 91 feet, the lower edge being 70 feet 6 inches below water and 50 feet below the bed of the river. The brickwork had all been built within two months and was laid in ordinary mortar. The cutting edge of the caisson was resting on a clay stratum into which the diggers had excavated holes about 9 feet below it, and as it was feared that the clay might at any time give way and cause a sudden sinking of the pier, it was determined to build up the brickwork higher than usual above the water level. The dredging was then recommenced, but immediately afterwards the new brickwork was burst outward by a sudden rise of the water inside the well, produced by the falling in of the clay.

The fracture was found to extend downwards to within 3 feet of the top of the caisson, or 21 feet below the river bed, and a mass of brickwork 62 feet in height by 46 feet in width was detached from the well and leaned outwards with

a slope of 1 in 15, while the channel-iron crating built up in the brickwork had been torn through from top to bottom.

However, after the rains it was discovered that the detached brickwork had fallen away, and the proposed blasting was thus rendered unnecessary. The shield was therefore constructed at the bridge shops and was lowered by winches from a staging erected over the gap, the sinking of the shield through the bed of the river down to the base of the fracture being done by dredgers working inside it. After properly cleaning the surfaces of the brickwork the shield was filled in with cement concrete, and the top of the patch was strongly bonded to the old work by rails. The well was afterwards sunk 70 feet 6 inches deeper, and is now the deepest foundation in the world.

Pier No. 1 was sunk only by hand excavation, being entirely in clay, and therefore easily kept free of water. In the case of this pier the weight of the pier superstructure caused a settlement of nearly 3 inches in the well, and the plan was then adopted of filling in the lower part of the caisson up to the top of its conical portion with clean sand before depositing concrete. The sand settled more closely under the slope at the base of the caisson than could be the case with concrete and much reduced the settlement in the other piers.

The weight of a main pier with its full load is approximately 16,000 tons and the area of its base is 1,430 square feet. This gives a pressure of 11·19 tons per square foot, but the effective pressure may be considered as reduced to one-half of this amount by water displacement and side friction.

The main piers are 72 feet 6 inches in height above low water. The well foundation is built up to 5 feet above low water and finished off with a rough-faced stone plinth; on this the pier is built, elliptical in plan similar to its foundation but having its major and minor axes 3 feet shorter, thus giving 1 foot 6 inches to spare all round the pier to allow for inaccuracy in sinking. No pier is more than 6 inches out of its true position.

The cost of the main piers as they stand was Rs.757,988, which gives an average of Rs.108,284 for seven piers sunk an average depth of 102·17 feet below low water.

The weight of a pair of main girders is 491 tons and that of a main span complete is 746 tons. The dead weight placed on a main span, including permanent way and metalling in the road and footway, is 224 tons.

The weight of a pair of the smaller girders is 42 tons and that of a span complete is 127 tons, the total dead load being 79 tons.

A Three-hinged Concrete Arch Bridge over the Danube at Ehingen, Germany.
"Engineering News," 9th Jan., 1902, p. 35.

The river bottom here was found to consist of gravel, with pockets and layers of very compact sand, to a depth of 10 feet to 14 feet below low water, unlaid directly by bedrock. The mean depth of the river at low water is only about 6 feet to 8 feet. The piers are stressed to 75 lbs. per square inch at the springing lines of the arches and 60 lbs. where they rest on the rock foundation.

The depth of the layer of gravel and the large quantities of water flowing in it gave rise, in connection with previous experience in the gravel strata of that region, to the belief that the common method of excavating pits and resting the foundation masonry or concrete directly on the rock would offer difficulties on the

point of excluding the water in the present case. (Cement grout was pumped through $1\frac{1}{2}$-inch pipes 12 feet long.)

The cement everywhere had set in a satisfactory manner.

In accordance with these results the left abutment was founded on a mass of concrete formed in place by such injection of cement. After completing the foundations several holes were drilled through down to bedrock to test the penetration of the cement. The drilling showed either good concrete or solid masses of cement, except for a layer of hard sand up to 2 feet in thickness. This sand, however, was so hard that loads up to 1,000 lbs. per square inch made no impression on it, and it was allowed to remain in the foundation.

To found the two river piers cofferdams of sheet piling were driven to rock and made watertight by injection of cement through pipes driven around them. In the case of one of the piers pipes were driven inside as well as outside, with the result that nearly the whole mass in the interior of the cofferdam was cemented into a block of concrete. On account of some layers of sand, however, about half of this mass was broken out again and the pier regularly built up of concrete above the remaining conglomerate. At the other pier the cementing was carried out only around the outside of the cofferdam, making it perfectly tight. The interior was then excavated and the concrete of the pier built up directly on the rock. At the right abutment a clay soil was met with and the excavation to each was carried on directly.

The total cost of the bridge was about \$21,000. (From "Centralblatt der Bauverwaltung.")

L'Ascenseur hydraulique de Fontinettes. Par M. Gruson, Ingénieur en chef des Ponts et Chaussées, 1888. Vol. 1 of the "Annales des Ponts et Chaussées," p. 694.

The Iron Wharf at Fort Monroe, Va. By JOHN B. DUNCKLEE. Trans. Am. Soc. C.E., vol. xxvii, 1892, p. 115.

The iron wharf at Fort Monroe, Va., was built in 1888–9 in accordance with the provisions of Acts of Congress approved 4th Aug., 1886, and 10th Aug., 1888. The wharf is built on hollow cylindrical cast-iron disc and screw piles (the latter having wooden bearing piles) spaced 14 feet apart from centre to centre in each direction, and braced by two systems of horizontal bracing of steel and wrought iron. The floor beams resting on the piles, on steel I-beams, and the floor joist and planking are of pine lumber; a fender system of wooden piles and lumber surrounds the outer faces of the wharf. The floor of the wharf is 7 feet 11 inches above low tide. The area is about 63,500 square feet.

The wharf extends from the shore in a southerly direction to a depth of 20 feet at low tide on the edge of the channel of Hampton Roads. The extreme length of the wharf from the shore to the southerly face is 322 feet. The width at the shore, and for a distance of 42 feet out from the shore, is 56 feet, increasing to 58 feet at a distance of 154 feet from the shore.

Cast-iron Piles.—It was at first proposed to build the wharf on wrought-iron piles with cast-iron disks similar to those used in the construction of the Coney Island and other ocean piers along the coast. Upon making borings at the site, however, it was found that near the outer limit of the wharf area, in depths of 20 feet at low tide, the stratum of sand was but 6 feet in thickness, while below this mud was found to a depth of 50 feet or more below low tide. While the thickness of the stratum of sand gradually increased as the water shoals

it was not regarded as affording a sufficient or safe bearing for disk piles outside the 10-foot curve, particularly in view of the mud beneath and the shifting bottom above.

It was therefore decided to use in depths of more than 10 feet a hollow screw pile resting on and encasing a wooden bearing pile previously driven and cut off from 3 to 4 feet above the bottom. Owing to the required shape of the iron piles the use of wrought iron was impracticable and cast iron was therefore adopted as the material of construction. In a depth of 10 feet or less a cast-iron disk pile was used. The cast iron was a close-grained, hard, white metal, intended to be without uncombined carbon, and of a character which would best resist the action of salt water, not excessively brittle, and which would satisfactorily bear drilling. All castings were coated inside and outside with coal, pitch and oil, according to Dr. Smith's process as used for water pipe.

Disk Piles.—The disk piles are hollow cylinders of cast iron with an inside of 8 inches and an outside diameter of 10 inches, the iron being 1 inch in thickness. At the foot of each pile is a disk 3 feet in diameter and 1·5 inch in thickness, cast with and forming a part of the pile, the distribution of weight from the pile to the disk being secured by means of eight brackets or ribs 1 inch in thickness. The disk or base of the pile is provided with a 2-inch nozzle, as shown for the water-jet, the nozzle being cast with four 1-inch ribs bracing it to the under side of the disk. Piles intended for depths of from 7 to 10 feet, and which, if cast in one piece, would exceed 20 feet in length, were cast in two sections with flanges 20 inches in exterior diameter and 1·5 inch thick at the point of junction. The disk piles were sunk by means of the water-jet without special difficulty, except where logs or piles were encountered. The disks are about 6 feet in the sand.

P. 118. *Screw Piles.*—The screw piles are of the form shown in plates and were generally cast in three sections designated as the lower, middle and upper sections. The lower section, which rests on and encased the wooden bearing pile, is 8 feet long and 13 inches in interior diameter, the iron being 1 inch thick. At the top of this section is a flange 24 inches in exterior diameter and 1·5 inch thick. In the centre of this flange is a 4-inch opening. About 1 inch above the lower end of this section of pile are two screw-pile blades with a diameter of 32 inches. The blades are 2·5 inches thick at the junction with the cylindrical pile, this thickness diminishing to ⅝ inch at the edge of the blade. The middle section of the screw pile is 8 inches in interior diameter and 10 inches in exterior diameter; iron 1 inch thick. The lower flange is 24 inches in diameter and 1·5 inch in thickness and is provided with eight brackets all 1·5 inch thick. There is an opening 4 inches in diameter in centre of this flange corresponding to the opening in the upper flange of the lower section of the pile. The upper end of the middle section is provided with a flange 20 inches in exterior diameter and 1·5 inch thick provided with eight brackets 1 inch thick. This flange is cast with an opening in the centre of the full inner diameter of the pile (8 inches). The variations in the depth of water were provided for by varying the lengths of the middle section by multiples of a foot, the length ranging from 7 to 17 feet. The upper sections are all 6 feet 9 inches long.

The wooden bearing piles were of pine creosoted with 12 lbs. of coal-tar creosoting oil to the cubic foot. The piles were 10 inches in diameter at the smaller end, and portion to be encased by the lower section of the iron screw pile was worked down so as to be 12 inches in diameter and perfectly straight. Piles had to be driven exactly 14 feet centre to centre. The bearing piles were

driven to a refusal equivalent to 1 inch for a hammer weighing 2,200 lbs. and falling 15 feet. The length of the pile required was generally from 50 to 60 feet and the length of pile in the ground ranged usually from 20 to 30 feet.

The hollow iron piles were afterwards filled with concrete composed of 1 Portland cement, 2 sand and 3 pebbles. The concrete was intended especially to prevent the corrosion of the interior of the pile by salt water.

P. 124. Total number of cast-iron piles 331, at $4\frac{1}{2}d.$ per lb. The work was under the charge of Lieut.-Col. Peter C. Hains. The writer was the principal Assistant Engineer.

Some Notes on Foundation Experiences. By A. P. Boller. Trans. Am. Soc. C.E., vol. xxvii, 1892, p. 471.

The plant of the Bay State Gas Company, built in 1886, is located near the pumping station of the main drainage works of the City of Boston, on Dorchester Railway, and on original marsh land flooded at high tides from the bay. It was designed by the late Joseph Flannery, a leading gas engineer of Philadelphia. The gas-holder tanks of the plant about to be described, and to which the writer's relation was simply that of a contractor, are interesting from their magnitude, the speed with which they were constructed, and the manner in which the work was carried out. There are two tanks built of brick about 30 feet apart, each having an inside clear diameter of 152 feet with foundation footings sunk about 30 feet below the level of the marsh.

No borings had been taken, it being assumed that the material to be gone through was substantially of the same character as that met with at the pumping station, starting with the marsh mud, stiffening up as clay, and reaching the sand, which was strongly water-bearing, as would be expected. The enormous area occupied by each tank dictated large pumping capacity.

The mode of construction adopted was as follows. The site of the tanks and construction plant (covering some 2 acres) was first dyked off from the sea, which flooded the marsh at high tide. Immediately below the tanks a sump well 10 feet square was sunk and planked up, into which all drainage was to be carried. The pumping plant consisted of two 80-H.P. locomotive boilers and four Andrew centrifugal pumps, two 6-inch and two 8-inch discharge, forming a duplicate plant.

In sinking the sump much difficulty was experienced, about two-thirds the way down, from marsh gas. The sheeting in single lengths was maulled down as far as possible, when a small steam pile-driver was mounted and the sheeting driven clear down below the concrete footing.

This manner of putting in the bank walls required only a narrow excavation easily braced, reducing the demands on the pumps to a minimum.

Inside radius of tank, 76 feet. 30 feet from granite coping to footing of concrete (see p. 473).

Cofferdam by Boiler—see Arthur Kill Bridge.

The Haarlem Ship Canal Bridge. By Wm. H. Burr, Proc. of Inst. C.E., vol. cxxx, p. 220.

The requirements of such traffic are supplied by the two openings, one on each side of the centre pier, 104 feet 1 inch wide in the clear at mean high water level. The two approach spans which flank the swing span are each 100 feet long from

the centre of the pier to the centre of the abutment bearing, and are of the ordinary riveted lattice type. The length of the structure is thus 551 feet 2 inches from outside to outside of the abutment walls. The street traffic requires a width of roadway of 33 feet 6 inches between the curbs, and two footpaths 8 feet 3 inches wide, making a total width of 50 feet.

This portion of Manhattan Island consists in the main of a ridge of indifferent marble, unfit for any purpose except the coarser kinds of masonry, and nearly the entire length of the canal is cut through this rock.

A fine, freely-running quicksand of variable depth, in which were found a considerable number of boulders, covered the rock, while above the sand was found the river silt and mud and other sediments characteristic of the banks and beds of both Spuytenduyvil Creek and Haarlem River. Although no foundation bed was more than 43·5 feet below mean high water level, it was determined to use pneumatic timber caissons for both abutments and pier No. 1, to avoid difficulties due to the presence of boulders in the quicksand which might attend the use of a cofferdam. The foundation beds for piers No. 2 and No. 3 were portions of the practically uniform rock bottom, and a cofferdam was used for the former and an open caisson for the latter. The position of the street was such that piers Nos. 2 and 3 could be placed on the bottom of the canal, but the irregular slope of rock prevented pier No. 1 from finding lodgment upon the same level, and necessitated its being founded upon a shallow pneumatic caisson sunk to the sloping rock surface.

Abutments.—The foundations for the north abutment were first placed. The material rendered the sinking of the caissons simple, but a considerable amount of rock cutting in the working chamber was necessary. As the face of the abutment wall was more than 60 feet in length, two caissons 46 feet 6 inches by 26 feet 6 inches were used for the foundations of the two retaining walls, each of which was 40 feet long. To secure a suitable foundation bed on the irregularly sloping rock surface it was necessary to excavate the rock on that side of the working chamber which first touched it. The deepest excavation was 12 feet in the east caisson, which brought about $\frac{2}{3}$ of the area within the cutting edge on the rock floor made by the excavation. The greatest depth of the rock surface below the cutting edge on the down-hill side of the caisson was 5 feet and the average depth about 2 feet.

Pier No. 1.—The two portions of the pier required small caissons only, about 26 feet by 16 feet. As the two columns of masonry resting on these two caissons are subjected to the thrust of the arch in addition to their vertical loads, their caissons were carried down into the rock until their cutting edges reached a continuous support throughout their lengths.

Centre Pier.—The foundation for Pier No. 2, which is the pivot pier for the swing span, was formed within a cofferdam. The greater part of the foundation lies on the roughly level rock bed of the canal, but a portion on the east side reached over the natural slope of the surface. The entire foundation was covered with mud and silt, which floated in to a depth of 3 to 5 feet when the adjoining dam was washed away. This was cleaned off by dredging before the timber work of the cofferdam was floated into place. An annular surface on the foundation bed 8 feet wide and around its entire circumference, just inside the dam, was cleaned of dirt and loose material by a scraper to enable a close bond to be formed between the concrete and the rock bed.

The central portion of the bed was cleaned, but with a little less care than was devoted to the annular surface outside of it. There can thus be no danger of

scouring under the concrete by the strong tidal currents, even though the lowest portion of the cofferdam should entirely disappear.

The filling between the timber sheels of the cofferdam was composed of about equal parts of clay and gravel of all sizes, and no puddling, other than that resulting from dumping the material into water, was used. After the completion of the cofferdam the mass of concrete forming the foundation of the pier up to 14 feet 9 inches (below mean high water) was deposited in skips or buckets holding 1 cubic yard each.

Pier No. 3 (p. 227).—The foundation for pier No. 3 required different treatment, viz., an open caisson on an artificially levelled bottom. A bottomless box was built of 6-inch by 12-inch timber, 27 feet by 17 feet 6 inches and 4 feet high, to which were secured 8 inches by 8 inches timber at the corners and about 6 feet apart along the sides. These vertical pieces were capped by 10-inch square timber. The frame, with the bottomless box, was drawn into position and sunk to the rock. Concrete was then deposited in the box to a uniform depth of 2 feet, after the rock surface had been scraped and cleaned. A uniform surface of the concrete was secured by moving a rail over two others.

The open caisson, in which the masonry of the pier was to be built, was constructed in the water near the site. The platform bottom measured 24 feet by 14 feet 6 inches in outside plan and was composed of two layers of 12-inch square timber. The foundation of the two portions of pier No. 3 were identical. The two greatest abnormal pressures on the foundation beds are 5,220 lbs. per square inch under the centre pier and 4,700 lbs. per square inch under the eastern portion of pier No. 1. These pressures exist when the greatest possible moving loads are on the superstructure.

The masonry in the piers and abutments above the caissons and the concrete foundations is divided into two classes, viz., foundation masonry and finished masonry. The former having its upper limit one course below low water and the latter reaching upwards from the foundation masonry to the coping courses and newels. The foundation masonry consists of the best quarry-faced limestone ashlar.

The Hawkesbury Bridge, New South Wales. By CHARLES ORMSBY BURGE. Proc. Inst. C.E., vol. ci, 1889, p. 3.

At the site of the bridge, about 7 miles from the sea, the estuary of the Hawkesbury has a total width of about 6,600 feet and is divided into two channels by Long Island. It was finally decided to accept the tender of the Union Bridge Company of New York, who undertook to complete the bridge within 2½ years for the sum of £327,000. The accepted design consisted of seven spans of 416 feet each from centre to centre of the piers, the foundation for the latter being of steel encased in steel caissons, while the upper portions of the piers and the whole of the abutments are of masonry.

Foundations.—The borings showed a bed of mud extending to a depth varying from 60 feet to 170 feet below high-water mark and overlying the sand, the greatest depth of water being 77 feet and the range of tide 7 feet. The greatest depth of foundation occurs at pier No. 6, which is carried down 162 feet below water, this being, as far as the Author is aware, the deepest bridge foundation yet sunk.

The caisson for each pier is made with rounded ends. It is 48 feet long transversely to the bridge and 20 feet wide, splaying out in the lowest 20 feet so

as to form a tapered shoe which is 2 feet wider all around the bottom. In the centre line on the plans, with its length, are three wrought-iron dredging tubes 8 feet in diameter and 14 feet apart centre to centre. These are connected to the outer skin and to each other by strutting of T's and angles.

At the bottom the dredging wells splay out in a trumpet mouth so as to meet the outer skin and also each other in a strong cutting edge formed of heavy steel plates.

The method of sinking the caissons was as follows. The shoe, having been built on shore at Danger Island and provided with a timber false bottom, was floated into position and sunk to the bottom of the river by removing the temporary bottom and partially loading the caisson with concrete.

The caisson was then sunk through the mud by dredging the material from the bottom of the wells and by loading the space between the wells and the skins with concrete, more steel being built up as the caisson went down.

As soon as the structure was firmly in the sand the dredging wells were filled with concrete and the masonry was then begun at a level somewhat below low water.

The concrete was composed of one part of Portland cement, three parts of sand and six parts of stone, broken to 2½-inch gauge. The stone was what is locally known as Kiama blue stone, the material being mixed by Jamieson's concrete mixers, each passing through about 5 cubic yards per hour.

The concrete in the shoe was made stronger by the addition of ⅛ of a cask of cement per cubic yard.

The caisson for No. 5 pier was the first one started, and having undergone greater vicissitudes in its downward progress than any other was the last to be completed. The sinking was begun on the 9th December, 1886, and the foundation was only ready for the masonry on the 9th October, 1888.

Shortly after it had well entered the mud the caisson showed a tendency to work eastwards, that is to say, transversely to the direction of the bridge. Efforts to recover its position were first made by endeavouring to cant it eastward at the top, by excavating the eastern well in advance of the others, thus pointing the central vertical axis downward in the westward direction required, but this was not successful, as even when the eastern well excavation was 15 feet deeper than the western one, the cant was still westward. Dredging outside was then resorted to without effect, also dumping the excavated mud outside on the eastern end. When the caisson had reached 75 feet below the river bed, the divergence had amounted to 5 feet at the bottom and 3 feet at the top, the cant still continuing westward notwithstanding constant extra sinking of the eastern wells. The margin for lateral divergence allowed by the specification for the caisson was 2 feet.

A recovery of about 18 feet at the bottom was effected about this time; and when the west cutting edge first felt the indications of sand, the caisson, as had been expected, commenced a righting movement toward the vertical, but of course at the expense of still further lateral divergence at the top.

The contractors now began driving piles at the east end to sustain a cribwork, which was loaded with stone and was intended to form a buttress against the structure and prevent further movement. (This moved however and proved ineffective.)

The experiment having failed, a proposal was made to cease sinking as the caisson was well in the sand at 144 feet below high water of ordinary spring tides, and then it was proposed to rectify matters by sinking an additional caisson at the

west end. This addition was in plan, something in the form of a crescent with rounded ends, the concave side of the crescent being meant to fit the rounded west end of the original caisson. It was made of steel plates. Two wells were provided in the new caisson, one at each extremity, the space between being loaded with concrete and the bottom finishing in cutting edges similar to the original one. The ill-luck of the pier attended this effort also, for when about 28 feet from the bottom the wells caved in under the pressure of the mud on the one side and of the original caisson on the other, so that further sinking became impossible.

As the additional caisson could not now be got either down or up there was no alternative left but to commence the masonry at the west end at as low a level as possible, viz., 12 feet 6 inches under the original masonry level, and to corbel out with the aid of a cofferdam. This corbelling was carefully carried out with solid stones of 7 to 8 feet in length, with a 9-inch overhang in each course, and though adopted as a last resource, the centre of the column of masonry above, coinciding with the centre of the west girder, is well within the base of the original caisson, and the resultant line of the pressures of the piers and load passes very closely to the centre of the bottom foundation area.

The subcontractor for the work attributed the eastern divergence to the fact that the mud at the eastern end was more consistent than at the west, hence the material displaced by the splay at the former end did not cave in immediately but stood for a considerable depth without filling the cavity. The west displacement, on the contrary, being immediately refilled by the softer mud falling into it, greater resistance was felt at that end, and the caisson gradually tended towards the other end, where the chief resistance was only at its projecting cutting edge.

No. 6 was launched about 6 weeks after No. 4, and gave some trouble. Its site coincided with a sudden declivity in the bottom of the river. The caisson tended northward following the declivity and continued to do so even when the sinking had progressed to a great depth, and in spite of the fact that large quantities of stone were dumped to the bottom at the north side of the structure. The caisson took a skew position, though not to such a degree as to interfere with the correct location of the masonry pier upon it, at right angles to the bridge. When the caisson reached the bottom at 162 feet below high-water of spring tides it had a slight lean to the southward at the top, notwithstanding that its proximity to the northern bank of the river had afforded facilities for counteracting this tendency by anchoring the top northwards.

Being unable to cope with the general northern movement of caisson No. 6, the contractors had to get the consent of the Government to increase the span of the girders between the fifth and sixth piers by 4 feet 3 inches north, so that the width of span No. 7 remains the same as originally intended.

The maximum pressure on the base of No. 6 caisson, which is the heaviest, is about 4 tons per square foot.

The foundation work was carried out by subcontractors Messrs. Anderson and Barr, of Jersey City, U.S.A.

P. 8. There can be no doubt, the Author thinks, that, having in view the recorded experience of the Jubilee Bridge over the Hooghly (Min. of Proc. Inst. C.E., vol. xcii, p. 79), Poughkeepsie and Hawkesbury Bridges, this class of foundation is well suited for excessively deep work ; but this experience, especially at the latter work, points to certain essential features of design the want of which gave much trouble.

In the first place the bottom outward splay should be avoided if it is desired to

diminish skin friction in sinking, the increase of size in the caisson shoe should be made by an offset.

Secondly, as the wells or dredging tubes are the only means by which the descent in such large and deep caissons can be regulated, they should be so distributed in plan as to facilitate this object. The wells therefore should be at least 4 in number, and disposed in a diamond or quadrangular plan, so as to allow of each well acting separately for regulating the descent.

Finally, notwithstanding these precautions, the proportions between the size in plan of the top of the caisson and that of the finished structure or pier to be raised upon it should be sufficient to allow of a moderate amount of deviation from the true location in the former without affecting the correct position of the latter.

Discussion on Indian Bridges. Min. Proc. Inst. C.E., vol. ciii, p. 151.

Mr. Harrison Hayter (p. 152): It would be noticed that the cylinders of the piers were of cast iron, excepting the bottom length, which was of wrought iron. This had resulted from experience gained during the construction of the Charing Cross branch of the South Eastern Railway across the river Thames. The cylinder piers of that structure were of cast iron from top to bottom sunk in the London clay; and notwithstanding that the bottom length was made thicker when a bed of spetaria was met with in sinking. This bottom length cracked in places, giving trouble and involving some additional cost. Since then, excepting only in the case of the Cannon Street Bridge, where the bottom was much thickened, his firm had always made at least the bottom length of the pier cylinders of wrought iron. In the Chittravati Bridge this bottom length was 3 feet deep and made sufficiently strong to absorb any strain that would come upon the cylinders during the process of sinking. It would also be noticed that the top length of the cylinders was an adjusting piece or cap of cast iron 2 feet 2 inches deep. Every one in the habit of sinking cylinders knew the importance of such a provision. Being of a larger diameter than the cylinder it could be moved up or down and bolted through to the cylinder exactly where required, forming at the same time a suitable projecting terminal cap to the column. This adjusting cap was filled with strong Portland-cement concrete carried up a little above the casting and splayed all round, so that the longitudinal girders would nowhere touch the casting but bear entirely on the concrete.

The average depth of cylinder sunk per working day was about 9 feet.

Mr. I. R. Mosse, (p. 163,) said that he knew of two instances in which foundations had not proved to be what was anticipated, without any blame at all being attributed to the engineers. The first of these occurred on the Intercolonial Railway of Canada, of which Mr. Sanford Fleming was engineer, about the year 1870. That railway in going through New Brunswick crossed a large tidal river at Niramichi by several spans of 220 feet each. The depth from high water to the sand being 30 feet. There was 10 feet of sand, then a bed of gravel 7 feet thick, and below the gravel 50 feet of silt. There was great discussion as to whether the foundations should be laid upon the 7 feet or whether they should go down to the rock through the silt, which would have made the depth of foundations from high-water level 97 feet. After a good deal of consideration Mr. S. Fleming determined to found upon the bed of gravel. The piers were of heavy ashlar work with large cut-waters to resist the pressure of the ice. They were put down in timber caissons 60 feet by 30 feet, and so much difficulty was experienced in getting the

foundation down to that bed of gravel that 1,416 cubic yards of material were removed from Pier No. 10 and 356 cubic yards of water were pumped out for every cubic yard of material. The piers were of solid masonry founded upon the bed of gravel over 50 feet of silt. They were loaded for 6 months with from 500 to 600 tons of rails. They all sank somewhat, but without cracks and with only a gradual settlement of the masonry. It proved very successful, and to the best of his knowledge no flaw has ever occurred. The minimum settlement was about 6 inches and the maximum 13 inches.

M. T. WRIGHTSON (p. 165): With regard to skin friction, it would not do to depend upon too low a coefficient. He had under observation some two or three years ago the case of two bridges in Devonshire across the Tavy and the other across the Laira, and in those bridges the cylinders went down into the mud 70 feet or 80 feet in the deepest part. One of the first works which the elder Rendel carried on was the building of the bridge across the Laira, and when his firm took the contract for a modern bridge within a few feet of that structure he consulted Mr. Rendel's Paper (Inst. C. E., vol. i, p. 99) with great interest to see what kind of foundation they would have to deal with. The design of the later work was made by Messrs. Galbraith and Church. When the cylinders were sunk into the river they had to weight them down, and a very curious thing happened. In many cases the weight had been on sometimes for a considerable time, when the cylinders suddenly sank for 10, 20, 30, and even as much as 40 feet. In the case of the Laira there was one that went as far as 42 feet in a few seconds, and in the Tavy there was one within 1 foot of that figure. The majority of the cylinders in both bridges sank in that way. He had made an estimate of the amount of skin friction which was overcome at the time when these runs occurred. In one case in the Laira Bridge, taking the weight of cylinder plus the weight of rail with which it was loaded, and assuming it to act over the whole of the subterranean part, the resistance amounted to 2·1 cwt., in another case to 2·5 cwt., and in another to 2·8 cwt. In the Tavy Bridge cylinders, most of which ran away, the skin friction was from 2·3 cwt. to 2 cwt., so that these figures approximately corresponded to Mr. Stoney's.

The Erection of the "Jubilee" Bridge carrying the East Indian Railway across the River Hooghly at Hooghly. By Sir BRADFORD LESLIE. Proc. Inst. C.E., vol. xcii, p. 73, 1887–88.

At the site of the Jubilee Bridge the River Hooghly is 1,200 feet in width at low water; this is the narrowest part for some distance above, while below the bridge the width is everywhere greater than at Hooghly. The site was selected partly on account of the small width to be bridged and partly for other considerations. The right bank is well above the highest flood level. On the left side the river is comparatively shallow, and there is a wide stretch of low ground which is inundated in the flood season, and which no doubt formed part of the main channel when the Damoodah River flowed into the Hooghly at Magra some five miles higher up. The deep channel on the Hooghly side is scoured out to the clay by the flux and reflux of the tide, while the depth on the left side of the river varies with the movement of the sand or silt which there forms the bed of the river. In the freshets a variable thickness of silt is displaced sufficient to adjust the sectional area of the river to the flood water to be

discharged. When the floods are strong the river bed is removed to a depth of 29 feet or more below datum.

The central double cantilever is 360 feet long and the two main side spans are each 420 feet long, or a length of 1,200 feet altogether, between the centre of the bed plates on the side abutments.

River Piers.—The site of the river piers is in about 27 feet to 30 feet of water at low spring tide in the dry season; the bed of hard yellow clay is met with at a depth of about 89 feet below datum at this part of the river, and the piers had to be sunk through about 60 feet of silt to reach the clay. The piers are 66 feet long, up and down stream, by 25 feet wide, with semicircular ends and flat sides. To sink these, two wrought iron caissons 108 feet high by 66 feet long by 25 feet with semicircular ends and each weighing 453 tons were provided. The caissons were divided into three excavating chambers, open from top to bottom, by two transverse compartments each 15 feet wide on the longitudinal axis of the caissons. These compartments afforded the buoyancy necessary for floating the lower portion of the caissons from the shore. To facilitate sinking through the river bed the lower portion of the caissons and of the transverse compartments was built with internal inclined surfaces extending from the bottom of the caisson to a height of 12 feet. The total area of the caisson was 1,516 square feet.

P. 74. The cantilever is carried at a height of 53 feet clear above datum on two river piers pitched 120 feet 6 inches apart from centre to centre. This arrangement had several advantages; first, it locates the piers in comparatively shallow water. It will be seen from figure in plate that if an ordinary bridge of three equal spans of 400 feet each had been adopted the pier on the Hooghly side would have fallen in 40 feet of water at low tide, a depth in which, knowing the force of the tidal bores and floods, the Author was not prepared to take the responsibility of pitching and sinking a large caisson.

P. 76. The lower 16 feet length of each caisson was riveted up complete on the river bank and launched on ways laid for the purpose, and when afloat it drew 9 feet of water. The upper portion of caisson was built up in rings or zones each 4 feet high and weighing 15 tons.

P. 78. The sinking of the caissons through the water was effected by the weight of additional caisson rings, brickwork between the shelf plates of the semicircular ends, and a certain amount of concrete deposited in the buoyancy compartments.

On the 26th April, 1884, just after all hands had left work, the No. 1 caisson on the west side of the river, which was afloat ready for pitching, drawing 32 feet of water and weighing 750 tons, was caught by the bore, which on this occasion, as is frequently the case, came up with a cant across the river and thus took the caisson at an angle with its axis. The caisson broke loose and was carried on the bore nearly ½ mile up the river and grounded across the stream, but on tolerably hard bottom. The caisson was recovered and finally was successfully deposited correctly in its permanent site.

The No. 2 caisson was pitched according to programme. Immediately a caisson was pitched the tide began to undermine it by scouring away the river bed outside, so that it was necessary to lose no time in starting to sink it by removing the earth from the inside.

To check scour outside when it threatened to overtake the sinking operations, bags filled with dense black clay were deposited in the holes scoured out; many thousands of these bags were used. Through the sinking having to be carried on

during the flood season, when there is no slack water for many days together, it was not an easy matter to fill the holes scoured out by the tide. It was safe to use stone rubble for fear of its working down under the cutting edge and impeding the sinking of the caissons ; the bags did occasionally so get drawn in, but did not check the sinking. On one occasion the scour so far undermined the No. 2 caisson when it was 60 feet high as to cause it to cant over to an angle of 1 in 8, and communication was established between the external water and the north boring compartment. The condition of the caisson was precarious. Heavy mooring anchors were laid out and connected by chain and rope tackles, "luff upon luff," to the top side of the caisson, so that an estimated force of 200 tons was exerted tending to right the caisson; then by boring steadily in the chambers, which were water-tight, the caisson gradually righted, the tackles being constantly set up to keep the maximum strain as the caisson came over. The righting of this caisson shifted the position of the cutting edge slightly so that the caissons were really 120 feet 10 inches apart instead of 120 feet 6 inches as originally pitched.

When the caissons got down to the hard yellow clay they stood firm on the cutting edge and did not penetrate it. The 10-foot holes excavated 8 feet in advance of the cutting edge, which were sufficient in sinking through the silt, had no effect. The boring gear had then to be lifted out of all the compartments, and two of the arms of each boring head extended so that they excavated holes $14\frac{1}{2}$ feet in diameter, and in fact to a certain extent undercut the transverse compartments. The excavation of these large holes in advance of the cutting edge caused the clay to break in and the caisson to sink. When the cutting edge was far enough into the clay to make it safe to remove water from the caisson, the level of the water inside the excavation chambers was lowered 24 feet by special pumps, which increased the effective weight of the caissons by 700 tons and helped to sink them through the clay. The caissons were thus got down until their cutting edges were over 100 feet below datum, at which depth, being firmly embedded 10 or 12 feet in the solid clay, further sinking was stopped, and the excavation chambers were filled to a height of 46 feet below datum with concrete of Portland cement and broken stone, and above that level with solid brickwork. The piers are of brickwork up to a height of 124 feet from the bottom of the caissons.

The total dead weight of one pier, including steel standard and superstructure, is 12,567 tons, and the maximum incidence of the moving load, consisting of two trains weighing one ton per lineal foot, is 1,320 tons, or together 13,887 tons ; deducting the frictional support of the river bed on the surface of the caisson at 5 cwt. per square foot or, say, 2,500 tons, the pressure on the base of each pier equals 11,387 tons, or at the rate of 7·5 tons on the square foot ; the normal pressure of earth and water at the same depth is 4·5 tons on the square foot.

To protect the piers from scour since they have been sunk, about 40,000 cubic feet of heavy stone has been deposited round each pier.

P. 128. The bed of clay (mentioned) was of great thickness, was impermeable to water, and in sustaining power and in resisting the action of running water was but little, if any, inferior to solid rock.

Description of the Iron Coal Pier, Norfolk and Western R. R. Co., at Lambert's Point, Norfolk, Va., and some of the Methods used in its Construction. By W. W. Coë. Trans. Am. Soc. C.E., vol. xxvii (1892), p. 125.

The river bottom at Lambert's Point is particularly well adapted to timber pile work and, incidentally, to iron piling. In the vicinity of the wooden coal pier the top stratum is a fine white sand varying in thickness from 1 foot to 3 feet. It has been packed hard by action of the water and offers considerable resistance to the penetration of piles, but is of considerable use in increasing its lateral stability. Underlying this sand we find a thick stratum of sandy marl through which the penetration of piles is comparatively easy, a pile 7 inches in diameter at the point being driven about 18 inches under a blow of a 3,000-lb. hammer falling 15 feet. The depth of this marl is slightly irregular but will average about 40 feet. Directly under this there is a stratum of sand on which all piles bring up well, and through which, in our construction at Lambert's Point, they have never been driven. It may be stated here, however, that from an artesian well boring near by it has been noted that the marl, with an occasional stratum of sand, extends to a depth of 1,200 feet, where calcareous rock is found.

The foundations for the pier consist of hollow iron piles in bents 36 feet apart, four piles in each bent. These piles, and, in fact all of the iron work, both sub- and superstructure, are protected from injury from contact, with vessels by having a timber protection of creosoted pine piles, which prevents a nearer approach than 8 feet.

The iron piles were made of wrought-iron piping 12 inches in diameter, ½ inch thick, in sections of from 14 to 20 feet, with flush joints and with a cast-iron disk 4 feet in diameter at the base of each pile. The transverse sub-bracing, composed of two lines of 12-inch channels 12 feet apart vertically, and connected with double angles forming a rigid frame, would then be placed one on each side of the bent. In sinking the piles, which varied in length from 45 to 57 feet, a top covering was placed on them and securely clamped by bolting, and a rubber hose connection 4 inches in diameter was made between the pump and the pile to be sunk.

A water pressure of from 30 to 60 lbs. was then put on, operating 10 jets at the bottom of the disk. One of these, 1⅜ inch in diameter, acted perpendicularly downward, and 9 ⅜ inches in diameter, worked on radial lines with a declination of 45°. The weight of the piles assisted by these jets would generally carry them down about 6 feet, when it was found necessary to apply blocks and tackle and to pull down with an estimated force of 35 tons, using the hoisting engine for this purpose. By this means the piles were sunk to the lower part of the marl stratum, and it was found impossible to pull them down farther. It was then decided to apply a test weight to each of the piles, using for this purpose a timber platform balanced on the piles and loaded with pig iron, the total weight being 64 tons. Under this there was settlement in nearly every pile varying from ½ inch to 6 feet in extreme cases, but averaging 13 inches for all. This settlement was undoubtedly due to the tearing up and softening of the bottom by the force of the water jet. The weight remained on each pile for four hours after settlement had ceased and the piles were then considered safe for any actual load that they might be called upon to stand. It might be proper to remark here that the pier has been in constant use by heavy engines and 60,000 lbs. capacity coal cars for nearly a year, and that no appreciable settlement has occurred. The average weight of the piles is 4,800 lbs., including the cast iron disks, and the total substructure weighs 736,000 lbs.

The Lansdowne Bridge over the Indus at Sukkur. By F. E. ROBERTSON. Proc. Inst. C.E., vol. ciii, pp. 123, 124.

The foundation work consisted simply in clearing away the material down to the rock. The abutments are of Portland-cement concrete.

La Louvière Hydraulic Canal Lift. "Engineering," February 24th, 1888, p. 201.

The following table gives the comparative capacities of the Anderton hydraulic canal lift near Northwich in Cheshire, and two others in course of execution on the Continent, the first at Fontinettes, upon the Neufossé Canal, in the department of Pas-de-Calais, France, and the second at La Louvière, on the Canal du Centre, in the province of Hainault in Belgium.

Name of Lift.	Anderton.		Fontinettes.		La Louvière.	
	Feet.	Inches.	Feet.	Inches.	Feet.	Inches.
Lift	50	2	43		50	6
Length of box between gates	73	9	132	10½	141	7
Width of box . . .	15	3	17	0	18	4
Depth of water in box .	4	5	6	6¾	8	6
Diameter of rams . .	2	11·4	6	6¾	6	6¾
	Tons.		Tons.		Tons.	
Weight to be lifted . .	250		770		1,100	
Displacement of largest boat lifted . . .	100		300		400	

The River Piers of the Memphis Bridge. By GEORGE SHATTUCK MORISON. Proc. Inst. C.E., vol. cxiv, p. 289 (date 1892–93).

The Memphis Bridge crosses the Mississippi near the southern limits of Memphis and is situated 232 miles below the junction of the Ohio and the Mississippi rivers at Cairo, Illinois. The authority for its construction was conferred by an Act of Congress, approved 24th April, 1888.

It is the first bridge across the real Mississippi. At Memphis the river feels the floods of both the Mississippi and the Ohio, though those contributed by the latter river are the more dangerous. The position of the river has remained unchanged for a long series of years at the site of the bridge, and as always happens in such cases with silt-bearing rivers, has become narrow and deep. The low-water width is about 2,000 feet, though the west shore line has been somewhat variable.

P. 290. The requirements of the Government chart fixed the minimum span at 600 feet in the clear, with a channel span of 700 feet. The Secretary of War required the long span to be next to the Tennessee shore, and indicated a preference for a span of 770 feet in the clear. The bridge as built crosses the river with one span of 790 feet 5 inches and two spans of 621 feet ¾ inch between centres of piers.

The three bridge spans rest upon four masonry piers. Two of these are near the low-water shore lines and may be properly termed shore piers. The other two are in deep water and are properly called river piers, and of these the Paper treats. The east pier is known as Pier II and the west pier as Pier III. The

plans of the piers were prepared in January, 1889. The data from which their character was decided upon were a set of borings which had been made two years previously, and such information as had been obtained during the sinking of the caisson of the west shore pier. Soundings had shown the bottom of the river to be 145 feet above datum at the site of Pier II and 160 feet at the site of Pier III. The borings had found clean river sand to 98 feet above datum, near the site of Pier II, and to 108 feet at the site of Pier III. At these levels clay was encountered, which at the higher levels was somewhat variable in quality, but into which one boring was made down to 61 feet above datum.

P. 289. (Low water, which is of rare occurrence, is 181·76 feet above the mean tide level in the Gulf of Mexico at the Government gauge, 2 miles above the bridge site. At the bridge site it is 181·6 feet, as nearly as could be determined. The highest water level that has been observed at Memphis occurred on 25th March, 1890, and was 216·2 feet above datum at the bridge site. All elevations given in the Paper refer to this tide-level datum).

This clay forms part of an immense deposit known as the La Grange formation, which is about 150 feet thick and perfectly watertight. Under this clay lies water-bearing sand or gravel, the outcrop of which is perhaps 50 miles east of Memphis and at a somewhat greater distance west of the city. The water-supply of the city is derived from artesian wells passing through the La Grange clay into the underlying water-bearing sand, and the wells which have now been driven afford valuable data as to the thickness of the clay. Ten of the wells sunk at Memphis have found the bottom of the clay at depths varying between 6 feet and 93 feet below the datum. A well situated 1½ mile west of the bridge found the top of the clay 85 feet and the bottom at 58 feet below datum.

It was evident that the foundations must be sunk through the sand and into the clay, and that whilst it was important to secure a thorough bearing everywhere on this clay it was probably so compact that difficulty would be experienced in any attempt to dredge it. Plans were therefore made to meet the four requirements: (1) the weight of the piers to be limited as much as possible; (2) provisions to limit the scour, at least during the construction of the work; (3) the base of the foundation to be large enough to keep the pressure within safe limits, even on a compressible clay; (4) the method of sinking selected to be the "plenum" pneumatic process.

To keep down the weights it was determined to build a high quality of masonry, to diminish the dimensions of the piers to a minimum and to build the lower portions of the piers hollow, a device which, objectionable in cold climates, seemed prudent here. The method of limiting scour was to carpet the bottom of the river with a woven willow mat, built floating on the surface of the water and then sunk in position by loading it with rip-rap. This device, which is believed to have been originated by the Author, proved successful. When the plans were prepared it was thought that a satisfactory bearing for Pier II could not be found higher than 83 feet above datum, but that a safe foundation for Pier III would be found 20 feet higher, or at 103 feet above datum. It was also thought that the bottom of the river could be maintained at the site of the piers at about 140 feet above datum. The general rule followed in determining the size of the foundation was to make the caissons of such construction that the weight of material below the bottom of the river should not be greater than that of the sand which they displaced, and that after deducting 400 lbs. per square foot for friction on the sides of the caissons the weight placed on the top of the latter should not produce a pressure on the foundation exceeding two tons per square foot. The base

required for such foundation was 92 feet long by 47 feet wide, and it was thought best to build the caissons with vertical sides below the bottom of the river.

The caisson for Pier II, with its upper works, was made 59·4 feet high and that for Pier III 39·6 feet high, both being alike in all respects, except in vertical dimensions. This was an error of judgment. It would have been wiser to have made them exactly alike and each 50 feet high. Good material was found at the site of Pier II, a few feet higher than was expected, and the square foundation projects above the bed of the river. It was necessary to go deeper with Pier III than with Pier II, and the masonry of this pier begins 10 feet lower than had been desired. The caissons are built of southern pine timber, most of which came from the State of Mississippi. The cutting edge is iron, of a form used by the Author at other bridges; this shape is preferred because it at once permits access to the actual edge when obstacles are encountered and provides a shoulder on which the caisson can bear when sinking through the same, the edge that projects below this shoulder preventing an influx of sand from without. The V-shaped walls surrounding the working chamber and the entire space between the timbers for a height of 17·3 feet above the bottom were filled with concrete after the caisson was placed in position. Above this concrete filling, for a height of 26·9 feet in Pier II, and 11·9 feet in Pier III, the interior portion only of the structure was filled with concrete, the outer parts being left empty, the upper 15·4 feet in Pier II, and the upper 10·4 feet in Pier III are of solid timber.

The larger caisson for Pier II contains 1,548 M.B.M. of timber and 424,000 lbs. of iron. That for pier 3 contains 1,078 M.B.M. of timber and 340,000 lbs. of iron.

The great depth to which the foundations went, as well as the fact that a considerable amount of clay had to be penetrated, made it important to provide special machinery for passing the men up and down and for removing the clay. Each caisson was provided with four 24-inch shafts for the removal of the mud, and these shafts were used to send in the concrete with which the working chamber was filled finally. Besides this there was one 36-inch shaft with a double air lock at the bottom of the pattern used on the piers of other works built by the Author; and one 6-foot shaft with a special air lock at the bottom and fitted with an elevator cage for the use of the men. Besides this the usual provision was made of pipes for air and water supply and for the removal of sand.

On the 4th of Sept. the foundation of Pier III was completed at a depth of 18 feet lower than had originally been intended. The masonry was completed on the 23rd of Jan., 1891. The increased height of the pier was made entirely in the masonry. The latter being correspondingly reduced. The whole of Pier II, including the masonry, was finished on the 25th of April, 1891. This foundation had been sunk 6 feet less than had been expected, and the difference required for correct dimensions was obtained by a slight offset in the masonry.

Both the foundations rest on clay, Pier II being sunk 13 feet into the clay, and Pier III 21 feet. In the case of Pier III the last 9 feet were of a very sandy character so that the foundation of the former is the better of the two. Both piers, however, rest on an entirely solid clay free from sand. (4 tests were made on clay.)

In behaviour the clay resembles rock rather than an ordinary clay and is capable of withstanding much greater pressures than the piers put upon it. It is also well adapted to resist scour. It is important to compare the strength of the clay as thus tested with the actual weights put upon it. The four tests made of clay from Pier II show an average strength of 13,400 lbs. per square foot when

entirely unsupported at the sides. The three samples tested from Pier III gave an average strength of 19,300 lbs. per square foot.

	Lbs.	Lbs.
The actual weight above the foundation of Pier II is, with the live load on the bridge .		10,410
Deduct for buoyancy below 182 feet above datum	3,722	
It is expected that at least 40 feet of this foundation will be perpetually buried in the sand; and to obtain the increased pressure on this area of foundation deduct the weight of this sand in water	2,320	
Deduct the skin friction on 40 vertical feet of caissons assumed to be 400 lbs. per square foot	1,029	
		7,071
Actual pressure per square foot of foundation . .		3,339

It would therefore appear that, making no allowance for buoyancy, the intensity of pressure on the foundation is less than that borne by the unsupported cubes experimented upon without material compression or the formation of shape; while the actual pressure, without allowance for skin friction, is less than ½ that borne by those cubes (2 cubes). Further, it must be remembered that this foundation is 40 feet below the bed of the river and that the mats around the piers will probably prevent scour near to the latter.

The pressures for Pier III are—	Lbs. per Square Foot of Foundation.	
Actual weight.		9,934
Deduct for buoyancy below 182 feet above datum	3,075	
Deduct for sand displaced	2,320	
Deduct for skin friction	1,036	
		6,431
Actual probable pressure . . .		3,503

The pressure on this foundation is a little greater than that on Pier II. As already stated, it would have been wiser to make both caissons alike and both 50 feet high instead of 40 and 60 feet.

The Foundations of the New Mutual Life Insurance Building, New York City. By T. K. Thompson. "Eng. News," 28th March, 1901, p. 221.

New building at Nassau and Liberty Streets, covering about 16,000 square feet. The first of these plans (accepted) placed the cellar floor on hardpan at an elevation of 55 feet with 6 feet side-walls of pneumatic caissons carried to an elevation 42 to 47 feet, as the only borings made at that time indicated rock or boulders at that elevation.

The contract was let to Messrs. Arthur McMullen and Company of New York City, on the 5th February, 1900.

Underpinning Cedar Street extension.

The first thing to be done after the old buildings had been removed was to take care of the adjacent buildings, and the first one tackled was the east wall of the old Cedar Street extension of the Mutual Life building. This was a first class brick (Portland cement) wall built on a 2-foot concrete base, 11 feet wide.

It was decided to support this wall by means of eight pneumatic caissons to be sunk to rock, spaced from 6 feet to 8 feet centre to centre. The caissons were made 36 inches outside diameter, the metal being 1½ inch thick, being 3 inches larger in diameter than the cylinders used by the same firm in 1896 in underpinning the east wall of the Stokes building, and 6 inches larger than those used in underpinning the Western Union building and Stock Exchange in the same year. The greater diameter of course made work in the air chamber vastly easier.

It was decided to allow a fibre strain of about 3,000 lbs. per square inch, which determined the thickness, 1½ inch, of the metal. Under the Stock Exchange the pressures were from 20 tons to 42 tons per square foot of base, under the Western Union they were from 30 tons to 70 tons. Under the Cedar and Liberty Street extension of the Mutual Life building we had about 36 tons per square foot of base, and the Stokes building we had in the worst case 56 tons per square foot. In figuring these loads on the base the friction on the cylinders, which amounted to about 450 lbs. per square foot of surface, or about 1½ ton per vertical foot, was neglected on the assumption that when we were sinking the caissons the material around the cylinders might be loosened up reducing the friction to almost nothing. Allowing for friction, the above loads would probably be reduced by about 25 per cent.

The bottom of the concrete foundation of this building was at the empirical elevation of 82·5 or 7½ feet above the elevation of standing ground water.

Cylinders always consisted of a bottom 4-foot section of steel plate, retaining cast iron for the upper sections, filled in with concrete (1–2–4).

Cylinder No. 1 was jacked through 42 feet of quicksand and 10 feet 6 inches of hardpan in 9 days. Cylinder No. 2, sunk in 8 working days, penetrated 8 feet 3½ inches into the hardpan, which was good material. No. 3, sunk in 7 days, going through 40 feet of quicksand and 12 feet of hardpan. The cutting edge struck a large boulder and the sinking was stopped. No. 4, in 9 working days, passing through 42 feet of quicksand and 8 feet 9 inches of hardpan, and eventually landing on a boulder which, when the big caissons were sunk, proved to be 7 feet deep. No. 5, after passing through 39 feet of quicksand and 15 feet 7 inches of hardpan, struck a layer of fine sand, boulders and decomposed mica 8 feet 1 inch thick before reaching the bedrock of N.Y. gneiss.

Before striking this soft stuff under the hardpan it was the intention to stop all the caissons big and little near the top of the hardpan, but with this treacherous material on top of the rock it was decided to carry all the permanent caissons to bedrock, although it more than doubled the amount of work originally intended and more than quadrupled the time required, as it took less than 2 days to put a caisson to hardpan and from 8 to 16 days to get it to bedrock.

No. 6 took 11 days work, the material being 40 feet of quicksand, 12 feet 7 inches of hardpan and 10 feet 10 inches fine sand and small amount of clay, finally landing on a bedrock of N.Y. gneiss.

No. 7 took 9 days, going through 42 feet of quicksand, 12 feet 10 inches of hardpan and 5 feet 5 inches of fine sand and boulders, finally on bedrock. No. 8 practically same as No. 7.

P. 225. The old cellars were on quicksand at or a few feet above the water level, which material continued to hardpan, which was from 30 feet to 35 feet below the water line. This was a very fine sand with more or less red clay in it at different elevations. Above water this material would stand vertically for perhaps 5 or 6 feet, but the slightest application of water would wash the whole

bank away; in fact it flows almost as freely as water when wet. There were practically no boulders above the hardpan. The hardpan, which varied in thickness from 8 feet to 15 feet, was very compact and difficult to remove with a pick; in some places there were no large boulders and in other parts of the lot there would be almost a solid mass of boulders.

In two cases there appeared to be hollow spaces in the hardpan of 2 or 3 feet capacity with absolutely nothing in them unless air.

While the material above the bottom of the hardpan was very uniform, the material below was exactly the reverse. At any point there was absolutely nothing between the hardpan and rock, while in other places there was from a few feet to 32 feet of fine sand, boulders and decomposed mica rock that could be cut with a knife. In nearly all cases the bedrock was what is known as N.Y. gneiss, with a horizontal stratification in some places and inclined or vertical in others.

Caissons.—A wall of 30 rectangular caissons 8 feet wide was placed around the lot as close together as possible, of four different lengths, viz., 8 feet × 18 feet, 8 feet × 17 feet 3 inches, 8 feet × 22 feet, and 8 feet × 15 feet 6 inches. All of these caissons had a ladder shaft and an excavating shaft each of 3 feet inside diameter.

Complete description of Stock Ramming given for making walls watertight.

Quantities.—2,600 tons of steel left in the foundation, as well as about 20,000 barrels of cement, 10,000 cubic yards of stone and 5,000 cubic yards of sand. Approximately 45,000 cubic yards of material have been taken out.

The architects of the building are Clinton and Russell of New York City, whose engineer is J. Hollis Wells. Mr. Alfred Noble was called in as Consulting Engineer. Arthur McMullen and Co., Park Row building, are the contractors for the foundations, and the writer is the engineer for the contractors.

Discussion on Railway Bridges. Proc. Inst. C.E., vol. ci, p. 38, 1889-1890.

Sir Bradford Leslie: The system of securing a good foundation in alluvial soils by sinking wells, caissons, or square blocks of masonry by excavating from the exterior, if it did not originate in India, had at least been more extensively practised there than elsewhere. It was applicable to foundations of any size and any shape, indeed the larger the area of the block to be sunk the more easy it was to keep it straight, and the less likely it was to be hung by side friction. It was generally preferable to use one large caisson or block rather than several small ones. Where several caissons were sunk in close proximity, one sometimes disturbed the others, and if they had to be sunk to any considerable depth they were apt to jam one another. In the case of the Blackfriars Bridge, there were, no doubt, good reasons for using three separate caissons for each pier, but for a very deep foundation a single caisson with three excavating chambers would have been preferable. With a single caisson 90 feet long there would have been no risk of tilting up or down stream, and it was probable that lateral canting would have been more easily controlled. All such caissons or block foundations in India, whether cylindrical, oval or rectangular in plan, were made with a vertical external surface, experience having shown that the vertical sides acted as a guide for the caissons and disturbed the ground less than any other form; the difficulty met with in keeping the caissons of the Hawkesbury Bridge straight was conclusive on this point.

The dredgers commonly used baled out a quantity of water at each lift faster

than the water could percolate through the bottom, and by degrees reduced the level of the water inside the caisson considerably below the level of the water outside, and this difference of level might be increased by the incoming tide. When the difference became very great it caused a blow-in below the cutting edge of the caisson and a large quantity of mud or silt was carried in from outside leaving a cavity outside towards which the caisson tended to travel. The stuft carried in was not always from below the caisson but from the side, and when once such a weak place was established it had a tendency to increase with each succeeding blow-in.

The best plan of avoiding such blows was to keep the water inside the caisson at all times level, or nearly so, with that outside, and to depend upon dead weight for sinking. In that way the caisson went down gradually, and the quantity of stuff excavated was limited to the net amount displaced by the caisson.

In sinking through clay or any material more or less water-tight, the lowering of the water level in the caisson rendered the portion of its weight that would otherwise be waterborne, effective for overcoming side friction, and greatly facilitated sinking ; but any considerable lowering of the water level in permeable strata was at the risk of sudden eruptions of mud and water from outside, causing the caisson to tilt and travel laterally. The steel caissons of the Hawkesbury Bridge appeared to have extended, with dredging tubes, complete up to the level of low water, and this was the case with the caissons of the Jubilee Bridge. At the Benares Bridge the caissons (iron) were of various heights according to the depth of water in which they were to be pitched. The weight on the foundations of No. 6 pier of the Hawkesbury Bridge was stated to be 9 tons to the square foot. This, it was presumed, was without allowing for any support from side friction, which, considering the unstable nature of the mud, could not amount to much. The area of the 2-foot splay at the base of the caissons, 245 square feet, must, however, carry the weight of the superincumbent mud and water, equal to 7 tons to the square foot, or 1,715 tons on 245 square feet ; this increased the average pressure on the total area of the base 1,134 square feet by 1½ ton to the square foot, making it 10½ tons altogether. At the Benares Bridge the weight on the base of the piers was 11·19 tons to the square foot ; at the Jubilee Bridge, disregarding side friction, it was 9 tons to the square foot, and at the Gorai Bridge it was 8½ tons. Such loads were sometimes objected to as excessive, and compared with the 4¾ tons to the square foot on the foundations of the Blackfriars new railway bridge, they appeared to be so, but it was simply a matter of depth. If the foundations of the latter bridge had been 100 feet deeper the weight on the foundations would have been increased to 9¾ tons to the square foot at least. As exemplified in the case of the Hawkesbury Bridge, very little advantage was gained by splaying out the base of a deep foundation because the area so gained had to support the normal weight of the superincumbent earth and water.

Surface Friction. By Mr. C. P. HOGG. Inst. C.E., vol. ciii, p. 190.

Friction per square foot of embedded surface depended not only on the nature of the strata, but even to a greater extent on whether the cylinders were exactly vertical, for the greater the deviation from the vertical the greater would be the friction per square foot. In the Alloa Railway Bridge across the Forth, erected in 1882–84, there was nearly 2,000 lineal feet of cylinder sinking. The cylinders were

5 feet, 6 feet, and in some piers 8 feet in diameter. During the progress of the works several good opportunities occurred for accurately observing the surface friction, and the engineers, Messrs. Crouch and Hogg, found it to vary from 2 cwts. to 5 cwts. per square foot, the higher rates being observed when the cylinder was out of the vertical or on resuming work after the operations had been suspended for several weeks. One of the 8-foot cylinders was sunk 74 feet below the river bed through the following strata :—

	Feet.
Silt and sand	2
Muddy sand, clay and stones	14
Sandy mud and stones	14
Running sand, mud and stones	17
Hard sand, stones and clay	2
Blown sand	14
Clean sand	9
Hard gravel, sand and clay	2
Total	74

At the finish the surface friction was 2·37 cwt. per square foot of embedded surface. In the observations made at Alloa Bridge there was nothing to show that under similar circumstances the surface friction increased per square foot as the depth increased. This had been quite confirmed by observation made during the sinking of the caissons of the Dalmarnock Bridge, just completed by Messrs. Crouch and Hogg, across the Clyde at Glasgow. The caissons for the piers of Dalmarnock Bridge were constructed of wrought iron, and were sunk by the pneumatic process from 50 feet to 55 feet through fine muddy clay and sandy mud. They were of oblong form, with parallel sides and semicircular ends, the dimensions at the cutting edge being, length 63 feet and width 9 feet, and at 56 feet above the cutting edge, length 62 feet 3 inches and width 8 feet 3 inches. On account of the large area of embedded surface the observations were of considerable value. The net sinking weight in the table was the weight of the caisson, concrete, air locks, etc., minus the lifting force due to the air-pressure in the working chamber at the moment the caisson began to sink. The values of the surface friction were probably somewhat high, as the caissons were slightly twisted.

Table of surface friction as deduced from observations made during the sinking of the caissons of Dalmarnock Bridge, Glasgow, by the pneumatic process.

Caisson.	Depth of the Cutting Edge below River Bed.		Area of the Embedded Surface of Caisson.	Net Sinking-weight = Weight, etc.	Surface Friction per Square Foot of Embedded Surface of Caisson.
	Feet.	Inches.	Square Feet.	Cwts.	Cwts.
No. 1.	38	9	5,251	18,974	3·61
	46	6	6,301	24,674	3·92
	49	5	6,684	25,754	3·85
	53	5	7,211	25,754	3·57
No. 2.	47	1	6,380	22,594	3·54
	53	0	7,155	24,640	3·44
	54	1	7,301	24,640	3·37

The Foundations of the River Piers of the Tower Bridge. By GEO. EDWARD WILSON CRUTTWELL. Proc. Inst. C.E., vol. cxiii, p. 117.

This Paper is limited to a description of the foundations of the two river piers of the Tower Bridge, and does not deal with the superstructure, which is still under construction.

The bridge has three spans; the middle one is 200 feet in the clear and those on either side of it 270 feet each. The middle span is constructed to open on the bascule system, to admit of the passage of vessels through the bridge, the waterway being spanned by two movable platforms each projecting 100 feet beyond the pier faces. Above the movable roadway is a pair of fixed footways, accessible by stairs and hydraulic lifts, and placed at such a height as to allow the movable platforms to be raised into a vertical position. The weight of the opening roadway, added to that of the high-level footways and the towers supporting them, renders the load upon the foundations unusually heavy for a bridge of such moderate spans, so much so that for a load of 4 tons per superficial foot the dimensions of the foundations worked out to 100 feet in width by 204½ from end to end of the cut-waters.

It was essential to adopt caissons of some kind for laying the foundations, because timber cofferdams were specially prohibited by the Act of Parliament. The ground to be passed through was London clay, the reliable nature of which renders it possible to effect a considerable saving by contracting the limits of the caissons within the outside line of the foundations, the full dimensions of the latter being attained by undercutting beneath the caissons to the extent of 5 feet horizontally. Thus the outside limits of the caissons measured 90 feet across by 194½ feet in length. Instead of sinking large caissons extending right across the pier, a system of smaller ones around it was adopted. There was a row of four caissons, each 28 feet square, on both the north and the south sides of the pier, and at each end of these rows was a pair of triangular-shaped caissons formed approximately to the shape of the cut-waters. The spacing between all the caissons was 2 feet 6 inches, this dimension being adopted as the minimum in which workmen could be effectively employed. The caissons enclosed a rectangular space 34 feet by 124½ feet, which was not excavated until the permanent work forming the outside portion of the pier had been built continuously within the caissons, and in the narrow spaces between them, to a height of 4 feet above Trinity high-water level.

The Caissons (p. 119).—The caissons were in two portions, the temporary caisson 38 feet high. Each portion consisted of a single skin of wrought-iron plate ½ inch thick at the bottom of the permanent caisson and diminishing to ¼ inch at the top of the temporary caisson. The cutting edge was of rolled steel, weighing 25 lbs. per lineal foot. It was riveted on the outer side of the skin, projected ⅝ inch beyond the latter, and formed in its descent a passage through the clay somewhat larger than the caisson above. The friction was thus reduced and the descent of the caisson consequently facilitated, and at the same time the natural swelling of the clay made good the space above the cutting edge sufficiently to prevent the water from passing down the outside and so into the caisson. The cutting edge was stiffened every 3 feet by vertical rolled iron joists, which were in turn supported by [two horizontal frames of 15 inches pitch-pine balks with diagonal struts of the same material at each corner, etc. (p. 149). No batter was given to the caissons.

Excavating inside the Caissons (p. 122).—The material excavated was London clay, covered in places with about a foot of ballast. It was compact and of uniform texture and so tough that, after sinking the caissons some 4 feet or 5 feet into it, the water could be pumped out and reliance placed upon its tenacity to prevent the water from the river forcing its way beneath the cutting edge. To permit the free rise and fall of the tide within the caisson in order to prevent any inequality of water pressure from forcing a passage underneath it, a sluice 9 inches square was provided near the top of the permanent, and another at the bottom of the temporary caisson.

(P. 125.) The weight of a square caisson, including timbering, was 166 tons, and that of a triangular caisson 207 tons. The greatest weight of the kentledge added was 274 tons in the case of a square caisson at the north pier, and the least was 86 tons for one of the triangular caissons at the south pier. For the square caissons the average weight of the kentledge was 208 tons at the north pier and 131 tons at the south pier, and for the triangular caissons 102 tons at the north pier and 92 tons at the south pier. (Concrete of Portland cement and Thames Ballast 1 to 6.)

(P. 132.) The piers from the river bed upwards are faced with rough-picked Cornish granite in courses between 2 feet and 2 feet 6 inches in height. The interior is built with wire-cut gault bricks, except the part that supports the opening span and the inside facework, which are of Staffordshire brindle bricks. The first course of granite was set on a bed of brickwork 2 feet thick over the whole surface of the concrete and extending level with the tops of the permanent caissons.

Conclusion (p. 135).—The erection of the first caisson was commenced in September, 1886, but it was not until January 1890 that both the piers were completed to the limits of the contract.

The total cost of the two piers to a height of 4 feet above Trinity high water, including all temporary works, amounted to £111,122.

Discussion on Preceding Paper (p. 145).—By Mr. J. WOLFE BARRY (p. 146).—He had considerable experience as to what that formation would bear. One of his earliest works was connected with the Charing Cross Bridge where the total pressure on the London clay was about 7 tons per square foot. At Cannon Street Bridge it was considerably reduced, being 4½ or 5 tons per square foot, but in both those bridges subsidence had occurred. It was not serious, but perceptible. In the Tower Bridge he wished to be on the safe side and reduced the limit of load rather below what it had been in previous bridges. He believed calculations showed the unit of load to be something under four tons per square foot. The amount of reduction of weight due to a part of the pier being always in water was not serious because the bulk of the pressure which came on the foundations was not due to the substructure, which was the matter now under consideration, but to the very heavy weight that came on the pier above the substructure.

The Erection of the Walnut Tree Viaduct on the Rhymney Branch of the Barry Railway. By ALFRED PEARCE. Proc. of Inst. C.E., vol. cxlix, p. 201, 1901–1902.

In the following Paper the Author describes the construction of the Walnut Tree Viaduct of seven spans which carries the Rhymney Branch of the Barry Railway across the Valley of the Taff, about 5½ miles north of Cardiff.

Foundations.—The excavation for the foundation of the piers was commenced on July 7, 1897, pier No. 2 being the first started. Owing to the nature of the soil passed through, which was in every case ballast, with thin layers of running sand, the excavation was carried down the full size of the concrete foundation, with timbering, runners being used in all cases. At pier No. 5 a cofferdam had to be constructed as the foundations of this pier were partly under the bed of the river. Piers No. 3, 4 and 5 were founded on ballast, and No. 1, 2 and 6 on compact mountain limestone rock; the rock being sidelong, and dipping towards the centre of the valley, was stepped to receive the concrete.

The concrete was brought up to within 5 feet of the surface of the ground and on it the brickwork of the piers was commenced.

Abutments and Piers.—The whole of the abutments and piers are built of bricks, the face being of Catty Brooke brindles and the inner work of red bricks made in the district. Owing to the methods employed in erecting the superstructure and in bringing up the piers, the brickwork had at times to withstand a pressure of 20 tons per square foot on work 7 days old. In the design of the piers in plan, as shown, two pockets are taken up to within 11 feet of the girder beds and are left hollow in order to lessen the weight on the foundation as much as possible, in no case being more than 4 tons per square foot.

The Weehawken Elevators and Viaduct. By THOMAS E. BROWN, Jr., and GEORGE H. BLAKELY.

This structure was constructed as a part of a general plan to provide rapid transit for the northern part of Hudson County, N.J., by the North Hudson County Ry. Co., which owns and operates all the surface and elevated lines of railway north of Pavonik Avenue in Jersey City. It is situated in Weehawken, opposite foot of West 49th Street, N.J. City.

Structure is designed to carry a double track standard gauge railway to connect the surface and car lines on the summit of the Palisades with the West Shore ferries.

(P. 3.) The structure as it now stands, both elevator plant and viaduct, was constructed from the plans and designs proposed by Thomas E. Brown, Jr.

Substructure.—The piers for towers, with the exception of those for the elevator tower and the tower adjacent to it on the west, rest upon rock. The foundations for these two towers were secured by means of piles driven to bedrock. The west abutment consists of a pocket blasted into the igneous rock of the Palisades, and levelled up with concrete.

Each corner pier of the elevator tower rests upon forty-nine piles arranged in a square of seven rows of seven each, 3 feet centres, driven to bedrock, which was at an average depth of 70 feet below the surface. The piles were sawed off 4 feet below low-water line and capped with two rows of 12 inches × 12 inches timbers laid at right angles. All timber in the foundations is kept below the

low-water line. The piers for the elevator tower are of concrete in four masses of full size, and of 3 feet thickness each, upon which rests a granite cap 6 feet square and 15 inches thick. Each pier is tied in either direction to the other piers with two tie-rods $2\frac{1}{4}$ inches in diameter.

(P. 5.) The maximum load upon each of the corner piers of the elevator tower is 628 tons, or an average load of 12·8 tons upon each pile. The maximum load upon each of the piers for the intermediate towers is 470 tons, or an average load of 15·6 tons per pile. The above loads are based upon the maximum combined effects of live and dead loads and wind, together with the weight of pier, which is taken at 148 lbs. per cubic foot.

(P. 4.) The piers for tower No. 2 each rest upon thirty piles arranged in five rows of six each, 3-foot and 2-foot 6-inch centres, capped in the same manner as those for elevator towers. Bedrock was reached under this tower at an average depth of 30 feet below the surface.

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ATIONS,

LL.

L. W. = Low Water.

		17–18	19	20	21
nking.	Char	Settlement.	Date when Information Given.	Authority.	Remarks.
issons		.. ..	1902	Wm. M. Patton. Foundation, pp. 294 and 308.	Sudden rises of 35 to 40 ft. occur in river.
ons		no settlement rrounding soil up	1873 1884	Wm. L. McAlpine ” ”	Trans. Amer. Soc. C.E., vol. ii, p. 287. Van Nostrand's Eng. Mag. ii, 242.
issons	3 p Pie	.. ..	1894	Geo. S. Morison	Report (final) pp. 4–6 and Tables.
”	Ha	.. ..	1883	Geo. S. Morison	Final Report, pp. 8–11 and tables. H.W. mark 1636 above datum. L.W. ” 1616 ” ”
”		t each of 3 spans, s. Moving load er 1 ft. for 999 ft.	1886	Geo. S. Morison	Final Report, p. 4 and Tables
..		after completion d strengthening	1903	J.R.Worcester, Eng. Sec. vol. xxx, p. 297.	Load eccentric about 3 ft. from centre slack.
ng	All	ned 30 tons without ettlement uniform, er 1 in. after 11 m.	1903	Witham Parker	Jour. Ass. Eng. Soc. fol. 30, p. 327.
issons		.. ..	1891 1893	G. S. Morison, Final Report, pp. 10, 12 & Tables	Caissons had vertical sides, Wm. M. Patton. Foundation, p. 292.
and aisson		.. ..	1890	Eng. News, ii, 115	Caisson lost 8 July, 1891.
ns	R	.. ..	1891	Eng. News,	i, 462.
..	La	.. ..	1889 1902	Eng. News, Wm. M. Patton,	ii, 352. 349.
..	Clay ft. b	enly. Required pairs	1893	Eng. News,	ii, 405.
tion	Clay, ft., th about	91, to July, 1902, nt. 14½ ins. from 902, no settlement	Jan. 1903	E. C. Shankland	Building 113 ft. 3 ins. × 164 ft. 9⅝ ins. on ground, by 302 ft. high.
..	Qui 1	niformly 5 ins., ed 6 ins.	1892	Ossion Guthrie, Eng. News, 92, ii, 345	No cracks appeared in building.
..		hs' settlement of 1 $\frac{3}{16}$ in. one ⅝ in.	1893	Corydon T. Purdy	Eng. News, ii, 486.
s, ner	H	.. ..	1898	Eng. Record, ii, 557	Piles cut off 12 ft. 5 ins. below L.W. level lake.
..		oad, 50 tons per fortnight. No lement.	1893	N. E. Weydert, Eng. News, 1893, ii, 3	Tested ability to carry 30 tons per pile safely.
tion	Sti	one	Jan. 1903	John Ericson	Stack 9 ft. internal diameter, 175 ft. high above ground.
..		formly 2½ ins.	1892	Ossion Guthrie, Eng. News, 92, ii, 345.	
..		in. uniformly	1902 1892	A. Gottlieb, Eng. Record,	Patton's Foundation, p. 372. i, 15.
and aisson	Rock No. 5 s	.. ..	1892	Gustave Haufman & F. C. Osborn	Caisson had batter of ⅛ in. per ft. Trans. Am. Soc. C.E., vol. xxvii, 173.
sson	1 ft. 9 in	al total load, less of displacement	1890	Wm. H. Burr	Trans. Amer. Soc. C.E., vol. xxiii, 53.
ing	Coa	.. ..	1902 1905	J. K. Freitag Chas. E. Fowler	Ordinary foundations, p. 168.

ATION

nking.	and 18 tlement.	19 Date when Information given.	20 Authority.	21 Remarks.
hrough ipe	t noticed by eye	1879	Chas. Macdonald	Trans. Am. Soc. C.E., vol. 8, p. 227. 20 ins. between rows of 4 or 8 piles 16 ft. 8 ins. or 17 ft. 6 ins. apart.
ation	.. ..	1900	Chas. S. Gowen	Trans. Am. Soc. C.E., vol. 43, p. 469.
m	.. ..	1892 1888	A. P. Boller Eng. News,	Trans. Am. Soc. C.E., vol. 27, p. 475. i, 535.
ammer	.. ..	1892	J. B. Duncklee	Trans. Am. Soc. C.E., vol. 27, p. 115.
aissons	.. ..	1885	Wm. M. Patton	Eng. News, i, 83, 122, 228, 244, 262 and 274
ging	.. ..	1894	Eng. News,	i, 544.
er	.. ..	1892	Thomas E. Brown and G. H. Blakeley	Trans. Am. Soc. C.E., vol. 27, p. 1.
aissons	.. ..	1894 1895	W. H. Gahagan Hunter McDonald	Trans. Am. Soc. C.E., vol. 31, p. 587 Trans. Am. Soc. C.E., vol. 33, p. 171
	.. ..	1890	Eng. News,	i, 249 ; ii, 51.
so block 35 tons	.. ..	1892	W. W. Coe, Trans. Am. Soc. C.E., vol. 27, p. 125	Heavy engines and 60,000-ton cars gave no settlement in 1 year.
..	as far as is known	1903	R. A. Hale	Jour. Assoc. Eng. Socs., vol. 30, p. 341.
ging	.. ..	1887 1902	Mace Moulton Wm. M. Patton	Trans. Am. Soc. C.E., vol. 17, p. 111. Foundations, p. 364.
issons	N N	1894	G. S. Morison	Caisson's sunk on mattress, 240 ft. × 400 ft. Final Report and tables, pp. 8–16.
son	satisfactorily	1900	I. O. Baker	Masonry construction, p. 274.
average , 10 ft. t.	.. ..	1902 1885	Wm. M. Patton, Wm. M. Patton,	Foundations, p. 321. Eng. News, i, 210.
g	Spth of 100 ft. to t. or more	1890	Alfred P. Boller,	Final Report, pp. 12-23.
issons	.. ..	1900	Eng. Record,	ii, 273.
issons	26,000 tons ; total rea, 3,575 sq. ft.	1894 1896	Eng. Record, Eng. Record,	ii, 104. ii, 28.
mer	.. ..	1894	Eng. News,	ii, 526.
aissons, below	.. ..	1900	Eng. Record, ii, 157	Adjacent buildings not underpinned. No settlement detected.
issons	N	1878	W. A. Roebling	Trans. Am. Soc. C.E., vol. 7, p. 331.
issons	C g	1878	,,	
ation el of ink by form	Sed $1\frac{3}{4}$ in. in 18 hrs., re in the following grest was abandoned. No settlement for infollowing 30 hrs. due to pumping obably	Feb. 1903	Foster Crowell	Under his personal direction and observation.

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L. W. = Low Water.

	and 18	19	20	21
Sinking.	lement.	Date when Information Given.	Authority.	Remarks.
caissons	116,500 to 686,400 . cast cylinders ardpan or rock	1897	Eng. Record,	i, 427–493
lam	orm settled 0·085 g 6 days no settle- beginning above ad been penetra- t. through softer stratum	Feb. 1903	Foster Crowell	Undertaken to show minimum supporting power, rather than real limit. Under personal direction
..	.. .. ..	1898	Eng. Record,	i, 27
avation	.. .. ..	1900	Eng. Record,	ii, 419
caissons	.. .. ..	1897	Eng. News,	i, 13
dam	.. .. ..	1897	Wm. H. Burr	Proc. Inst. C.E., vol. cxxx, pp. 224–28
caissons	ent observed	1893 1894	Eng. News, ii, 458 Eng. Record, i, 122 Chas. Sooy Smith	Trans. Am. Soc. C.E., vol. xxxv, 459.
caissons	.. .. ..	1899	Eng. Record, vol. ii, 509	Heavy loading required to sink caissons, as much as 75 tons for 1
caissons	.. .. ..	1897	Eng. News,	ii, 38
caissons	.. .. ..	1896	Eng. Record,	i, 315
caissons	.. .. ..	1901	T. H. Thompson	Eng. News, i, 221. Underpinning of Cedar St. extension
avation	.. .. ..	1902	J. K. Freitag	..
..	.. .. ..	1894	Chas. Macdonald	Board of Engineers' Report, p. 56
caissons	.. .. ..	1894	G. Lindenthal	,, ,, ,, p. 74
n or coffer d and iro	.. .. ..	1894	,,	,, ,, ,, (following p. 74)
n dredgin	.. .. ..	1892	Eng. News,	i, 138
c caisson erdam	.. .. ..	1892 1893	W. Gustav Triest	Eng. News, i, 526 ,, ,, ii, 198
tion 34 ft curb	.. .. ..	1896	Eng. News,	ii, 226
avation	ns. square carried a fortnight; no n. total settlement 1,500 lbs.	1896	Eng. News,	i, 310
caissons	.. .. ..	1898	Eng. News,	ii, 363
avation	.. .. ..	1900	Eng. News,	i, 397
caissons	.. .. ..	1896	Eng. Record,	ii, 107
caissons	.. .. ..	1901 1901	Eng. News, Eng. Record,	ii, 222 ii, 289

L. W. = Low Water.

Met	17–18 Settlement.	19 Date when Information Given.	20 Authority.	21 Remarks.
O	of 520 ft. Live load sses, 11,800 lbs. r lineal ft.	1903	Eng. News,	i, 85.
Pne	.. .. ..	1902	Edwin Duryea in Eng. News, i, 358	Size of pier governed by lateral dimensions of Tower leg.
Pne	ortunity for tests Smith ever had	1873 1888	Wm. Sooy Smith Geo. S. Morison	Trans. Am. Soc. C.E., vol. ii, p. 411. Final Report, New Omaha Bridge, p. 3.
O	0 ft., 3—192 ft. span. , R.R., and live load ,000 lbs. per lineal ft.	1893	Eng. News,	ii, 410.
C	.. .. ..	1902	Wm. M. Patton	Foundations, p. 149.
Pne	.. .. ..	1886 1887	Wm. M. Patton H. T. Douglas	Eng. News, i, 85–195. Foundations, p. 130 & 294. Eng. News, i, 151.
Ste	il in early part 1877. th end settled 20 ins. Feb. 10, 1878	1878	D. McN. Stauffer, Trans. Am. Soc. C.E., vol. vii, p. 264	Piles were from 28 to 30 ft. long. Average length cut off 25 ft.
Pne	t whatever in any cyl.	1878	D. McN. Stauffer, Trans. Am. Soc. C.E., vol. vii, p. 272	Owing to faulty construction, part of weight is thrown on outside columns.
	.. .. ..	1878	,, ,,	
Op	.. .. ..	1902	Eng. News,	ii, 219.
Pne	le to scour to rock. ns. Single track bridge	1882	Geo. S. Morison	Final Report, pp. 4–8.
	.. .. ..	1902 1885	Wm. M. Patton ,,	Foundations, p. 132. Eng. News, i, 165, 338.
Op	.. .. ..	1888	John F. O'Rourke	Trans. Am. Soc. C.E., vol. xviii, p. 199.
Ope coff	.. .. ..	1878	Gen. G. K. Warren	Report on Bridging the Mississippi River, p. 1,011.
Pne	.. .. ..	1891	S. M. Rowe	Trans. Am. Soc. C.E., vol. xxv, p. 663.
Op	.. .. ..	1902	J. K. Freitag.	
Pne	.. .. ..	1889	O. Chanute & J. F. Wallace	Trans. Am. Soc. C.E., vol. xxi, p. 97.
	.. .. ..	1881	C. M. Woodward	History of St. Louis Bridge, p. 61, etc.
Pne	.. .. ..	1878 1871	C. Shaler Smith Van Nostrand	Trans. Am. Soc. C.E., vol. vii, p. 335. Eng. Mag., vol. v, p. 178.
	place old pier	1894	H. G. Helley	Trans. Am. Soc. C.E., vol. xxxi, p. 598.
Platfor ing	.. .. ..	Feb. 1903	Foster Crowell.	
H	tically none	Jan. 1903	Bernard R. Green	½ Portland cement concrete, ½ natural (Cumberland) ,,
..	.. .. ..	1900	I. O. Baker	Masonry construction, p. 191.
Pne	.. .. ..	1889	C. D. Purdon	Trans. Am. Soc. C.E., vol. xx, p. 151.

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L.W. = Low Water.

...thod	17 and 18 Settlement.	19 Date when Information Given.	20 Authority.	21 Remarks.
Hand	Practically none	Jan. 1903	Bernard R. Green	Foundations of Portland cement concrete.
"	...rpinning job. Settlement ...on, 2¼ ins., gradually and ...y during operation only	Jan. 1903	"	"
"	Practically none	Jan. 1903	"	"
...euma	.. .. ..	1904	Eng. Record	i, 333. Brooklyn and New York piers practically the same.
Open pile	.. .. ..	1870	O. Chanute, Geo. S. Morison	Final Report, pp. 33–72. Pier 2 carries entire weight of draw and turn-table.
...euma	...ght each through span, ...,800 lbs. Moving load, 3,000 lbs. p.l.ft.	1892	Geo. S. Morison	Final Report, p. 4, and tables.
...euma	... at span 973,600 lbs. and ...ng. 4 through spans 250 ft. ...ving load 8,000 lbs. p.l.ft.	1888	Geo. S. Morison	Final Report, pp. 4–5, and tables.
...euma	.. .. ..	1873	Geo. Wm. S. Smith	Trans. Am. Soc. C.E., vol. ii, pp. 414–417.
...euma	...ack R.R. bridge. 3 chan... ...ans at 375 ft. long and ...lbs. weight. Moving load, 3,000 lbs. p.l.ft.	1890	Geo. S. Morison	Final Report, pp. 4–5, and tables. Caisson batter, 1 in 24.
...euma	...ack R.R. bridge. 4 spans ...0 ft. and 1,110,000 lbs. Moving load, 3,000 lbs. p.l.ft.	1890	"	Final Report, pp. 4–6, and tables.
...euma	.. .. ..	1890	Wm. R. Hutton	pp. 19–22. Work on same.

L. W. = Low Water.

Met	17-18 Settlement.	19 Date when Information Given.	20 Authority.	21 Remarks.
	ı clay, tested 28 tons for 24 to 28 hours, ʾ settlement, 4 ins. 2¾, maximum, 10½ ins.	1902	Ernest M. De Burgh	Minimum test after 5½ weeks. Maximum test 12 hours after sinking.
	ıded with 18 tons for ys gave no settlement in. to 2 ins. ; 20 blows ʾ tons gave no effect. produced further set ı loaded piles.	1902	Ernest M. De Burgh	Proc. Inst. C.E., vol. cl, p. 340.
Op	.. .. ..	1890	Chas. O. Burge	Proc. Inst. C.E., vol. ci, p. 3.
..	.. .. ..	1897	Eng. News,	ii, 226.
O	.. .. ..	1887	G. H. Massy	Trans. of Canadian Soc. C.E., vol. li, p. 36.
..	.. .. ..	1890 1891	Eng. News,	i, 338. i, 524.
O	.. .. ..	1876	Sir Sanford Fleming	"The Intercolonial," p. 140–200.
	ıled during construction, s were applied. Settled Loading of 550 to 575 ı each pier produced ent about 6 to 10 ins.	..	.. .. ..	Same, p. 200.
..	.. .. ..	1873	B. B. Stoney, Theory of Strains, p. 501	Pressure calculated at base of Britannia Tower.
..	ran away 10 ft., 20 ft., 10 ft. suddenly	1890	T. Wrightson	Proc. Inst. C.E., vol. ciii, p. 166.
O	th load 50 % more than load without movement	1892	Francis Fox	Proc. Inst. C.E., vol. cviii, p. 304.
..	ran away 10 ft., 20 ft., d in one case 42 ft. in a few days	1890	T. Wrightson	Proc. Inst. C.E., vol. ciii, p. 166.

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L.W. = Low Water.

	17–18	19	20	21
Metho	Settlement.	Date when Information Given.	Authority.	Remarks.
Open	.. ..	1899	G. G. Lynde	Proc. Inst. C.E., vol. cxxxvii, p. 364
	owed sustaining power ut 5 tons per sq. ft.	1858	James Brunlees	Proc. Inst. C.E., vol. xvii, p. 442
Ope	.. ..	1873	B. B. Stoney, Theory of Strains, p. 501	Pressure calculated at base of middle pier
edgin pneur	settlement in use	1892 1892	Eng. News, Chas. Neate	ii, 224 Proc. Inst. C.E., vol. cix, p. 304
Ope	.. ..	1890	E. F. Chalton	Proc. Inst. C.E., vol. ci, p. 13
Ope	, on total area of pier 1 for every 20 ft. depth. ably safe to take 1 cwt. for every 10 ft.	1888 1902	Sir Bradford Leslie ,,	Proc. Inst. C.E., vol. xcii, pp. 73–141. Letter.
By d	ht of about 500 tons y required to sink wells	1901	G. H. Eves	Proc. Inst. C.E., vol. cxlv, p. 292. Designed against overturning.
)pen)neu	alues for 36 cylinders s from 17 ft. to 64 ft.	1890	Edward W. Stoney	Proc. Inst. C.E., vol. ciii, p. 135.
ylind y a ‘	acting within 18 ft. of of pier gave practically 6 tons	1903 1872	Sir Bradford Leslie ,,	Proc. Inst. C.E., vol. xxxiv, p. 1.
Ope	.. ..	1893	Eng. News,	ii, 49.
offer	.. ..	1905	Chas. E. Fowler	"Ordinary Foundations," p. 7.

L. W. = Low Water.

		17–18	19	20	21
ing.	Cha	element.	Date when Information Given.	Authority.	Remarks.
sons		importance	May 1903	M. Georges Hersent.	
..	Sol	.. .. ..			•
		l at depth of less at bearing power east 7¾ tons	1892	E. J. T. Manby	Proc. Inst. C.E., vol. cviii, p. 318.
sons		.. ..	1898	Eng. Record,	ii, 556.
sons	Tu	importance	May 1903	M. Georges Hersent.	
ssons		notes	Feb. 1903	A. Charguéraud.	
..		move for 32 hours. t by immersion at friction was over- Friction, 114 tons	1879	A. Schmell	Van Nostrand's Mag., vol. xx–cxxi.
ssons ms	C	.. .. ..	1898	Eng. News,	i, 254. Sides of caissons slightly oblique. 83° 38′.
ssons	Gra	.. .. ..	1902	Wm. M. Patton	Tower, 984 ft. high. Foundations, p. 348.
s		.. .. ..	1902	Eng. News,	i, 35. Grout pumped through 1½ in. pipes.
..		.. .. ..	1890	C. Riensberg	Proc. Inst. C.E., vol. ciii, p. 435.
piles	Cl	.. .. ..	1905	W. Tierney Clark	Fowler's ordinary foundation, p. 80.
..		.. article	1902 1904	Eng. News, Emilio Rosetti	Tower, 40 ft. square and 89 m. (292 ft.) high. La Ingenieria (Buenos Aires), 3/31, p. 63.
ssons	C de	notes	May 1903	M. Georges Hersent.	
ing	Eit gre c	.. .. ..	1902	A. Iolziarski,	Eng. News, i, 286.
ion, t piles	Gr	as estimated to 45 tons	1899	Eng. Record,	ii, 47.
ssons	Ir	.. .. ..	1892	Eng. News,	i, 356.